PERFORMANCE OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO EARTHQUAKE MOTIONS

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Table of Contents

Tables

Figures

Abstract

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Two simplified methods for estimating the performance of reinforced concrete structures subjected to earthquake motions were evaluated. Both the Flat-Rate and Target Period methods characterize the expected level of performance in terms of the maximum estimated drift for a given intensity of ground motion. Drift estimates using the Flat-Rate method are based on the area of structural members, the total floor area of the structure, and the peak ground acceleration as a measure of earthquake intensity. The main parameters for the Target Period method are the initial period of the structures and the peak ground acceleration.

The applicability of these methods to assess the expected level of performance of existing structures was investigated using experimental data. Drift values calculated with the Flat-Rate and Target Period methods were compared with measurements obtained from earthquake simulator tests performed on reduced-scale models of reinforced concrete structures.

Results indicate that both methods provided an adequate assessment of performance.

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Chapter 1

1.1 Introduction

During an earthquake, ground motion can severely damage buildings and equipment housed in them. Ideally, buildings constructed in regions with significant seismic risk are proportioned following earthquake-resistant design procedures which makes them able to resist earthquake motions without being severely damaged. However, the design procedures used in older structures might not meet the criteria of modern earthquake design, quality control during construction may not have been sufficient, or the design of structures built in low seismic regions might not have accounted for the additional forces. All of these questions raise concerns about the safety of these buildings. For this reason, procedures are needed to determine the ability of existing structures to withstand earthquake forces and their safety in the case of an earthquake.

Different methods are already available to determine the expected performance of a building subjected to earthquakes, but most of them require elaborate calculations and knowledge of the design details. If a large number of buildings has to be evaluated, implementing these procedures will be time consuming and expensive, and, in cases where design details are not known, it will be very complicated. Given the complexity of current methods, procedures that can be used to estimate the structural performance under earthquake loading and that can be applied if only the basic geometry of the structure, the member sizes and material strength is known are needed.

One possibility to quantify the performance of buildings subjected to seismic loading is to determine the maximum drift and to set a limit for this quantity. Two methods, the flat- rate and the target period method, have been proposed by Hassan and Sozen (1997) and Browning et al. (2000) to achieve this goal. Conceptual studies which indicated good results have been conducted by Browning using LARZ, a nonlinear analysis program developed at the University of Illinois (Otani, 1974, Saiidi, 1979a, 1979b, Lopez, 1988).

The goal of this study is to validate the proposed methods by applying them to 'real', existing structures. The problem of comparing the calculated and measured response of buildings is that there are very few structures that have been instrumented to record their response during an earthquake, and therefore there are not enough data available. An alternative solution to evaluate the proposed methods is to use data obtained from experimental analyses of small-scale models to verify the results of the two methods and their correlation with measured quantities. For this reason, data obtained from dynamic tests using earthquake simulators has been collected and used to evaluate the flat-rate and the target period method based on the measured response of the model structures.

The test models considered were built and tested between 1975 and 1997 at the University of California, Berkeley, Oliva (1980, 1987), Hidalgo (1975), the University of Illinois at Urbana-Champaign, Moehle (1980), the National Technical

University of Athens, Greece, Vintzileou (1998), the Building Research Institute at Tsukuba, Japan, Kitagawa (1984), the Ohbayashi-Gumi Institute in Tokyo, Japan, Eto (1980), and the National Research Institute for Earth and Science and Disaster Prevention, Japan, Minowams (1994).

Criteria for the performance under earthquake loading can be the allowable rotation, drift, or expected damage for a given structural element or system. As mentioned before, in this study, the maximum roof drift was chosen to quantify the level of performance and the results obtained by applying the simplified methods will be compared to the experimental results.

The test models will be described briefly in the first chapter. In the second chapter the two simplified methods, the flat-rate and the target period method, will be explained and derived. More detailed information about the test models, such as member sizes, scale factors, the test input and the results obtained by calculation and experiment will be shown in the third chapter and in the fourth chapter the results will be evaluated.

1.2 Test models

Most of the structures were simple frame structures ranging in height from one to nine stories. A frame structure tested by Vintzileou, Yong Lu and Zhang (1998) had a centralized structural wall.

Vintzileou et al. tested a variety of almost identical six-story, three-bay frame structures basing their design on the relevant provisions of Euro Code 8. In the current study, results from two of these structures will be used, one of them without walls, the other one with a centralized structural wall. The length scale factor used was 1/5.5, leading to a bay length of 2370mm and a total height of 3660mm. The 1940 El Centro record was used for the earthquake simulator excitation. Both structures were subjected to five different peak acceleration levels of 0.1g, 0.3g, 0.6g, 0.9g, and 1.2g and the corresponding displacements were recorded. The investigation by Vintzileou et al. focused on the seismic behavior of multi-story R/C frames with different types of irregularities.

The remaining test structures were divided into groups according to their number of stories. Minowams et al. (1994) tested two one-story one-bay full-scale models differing only in the density of the hoop reinforcement of the columns, to investigate the influence of ductility on the structural performance under seismic loading. The total height of the specimens was two meters and the bay dimensions were six by six meters. In this experiment, peak accelerations ranging between 0.64g and 0.82g based on the east-west component of the Tokachi-oki earthquake were used.

In 1975, Hidalgo tested a two-story one-bay 7/10 scale model and subjected the structure to base accelerations between 0.06g and 1.49g based on the Taft, El Centro and Pacoima records. In 1980, a similar structure was tested by Oliva. For this model, the bay length was 3658mm and the total height was 4426mm.

Kitagawa (1984) tested two identical two-story one-bay structures built at half scale. The total height of the models was 3375mm and the bay dimensions were 3000mm by 1000mm. Kitagawa used the Miyagi-ken-oki earthquake records and subjected the structures to peak base accelerations of 0.216g and 0.566g.

Two three-story two-bay structures built at a scale of 1/6 were tested by Eto, Hiroaki and Takeda (1980). The bay length was 110mm and the total height was 1690mm. The north-south component of the Tokachi-oki and the Taft S69E records were used and the models were subjected to peak accelerations ranging from 0.197g to 0.308g.

Moehle and Sozen (1980) tested a nine-story, three by one bay structure that was built at a scale of one to twelve. The bay lengths were 305mm and 914mm and the total height of the structure was 2286mm. The north-south component of the El Centro record was used in the tests, and the record was scaled to have peak accelerations between 0.32g and 0.78g.

Note: additional results reported by Browning et al. (2000) are included in the analysis. These include specimens tested by Bracci et al., Fillatrault (1998), Otani (1972), Lybas (1977), Wolfgram (1984), Van Nuys, Eberhard (1989), Healy (1978), Moehle (1978), Cecen (1979), Wood (1986), Aristizabal (1979), Abrams (1979), and Schulz (1985). A total number of 46 specimen and 161 tests are used in the evaluation of the methods.

Chapter 2

2.1 Flat-rate method

2.1.1 General derivation of the method

This method is based on the observation that in systems with a period greater than the characteristic period of the ground motion, Tg, the maximum nonlinear displacement can be bounded by the maximum displacement of a similar linear system.

The implication of estimating the maximum nonlinear displacement based on displacement in structures that do not have any abrupt changes in stiffness or mass, is that drift can be controlled by limiting the period of the structure. This can be done by reducing the flexibility to lateral forces or reducing the mass of the system. The flat-rate method and the target period method prescribe limits for the lateral stiffness and period of a structure in order to limit the maximum drift within an acceptable threshold, which is used to define the overall performance of the structure. For a linear system, the relationship between spectral acceleration, Sa, and spectral displacement, Sd, can be approximated by

$$Sd \approx \frac{Sa}{\omega^2} \approx Sa \cdot \frac{M}{K} \tag{1}$$

Sd = spectral displacement

Sa = spectral acceleration

ω = circular frequency of vibration

M = total mass of the structure

K = stiffness of the structure

The simplifying assumptions made for this procedure are that the mass M is proportional to the total floor area A_{fl} of the structure and that the stiffness K is proportional to the sum of the areas of the columns A_{ce} and walls A_{wt} in the base story.

Hassan and Sozen (1997) suggested the following equations to calculate the effective cross-sectional areas of walls and columns:

$$A_{ce} = \frac{A_{col}}{2} \tag{2}$$

$$A_{wt} = A_{cw} + \frac{A_{mw}}{10} \tag{3}$$

where A_{cw} = area of reinforced concrete structural walls

A_{mw} = area of masonry walls

For the purpose of calculating an equivalent floor area for specimens from earthquake simulator tests, the story weight was adjusted according to scale factors for length and time according to Eq. 4:

$$A_{fl} = SW \cdot \frac{SF_L}{SF_T^{\,2}} \cdot \frac{1}{\gamma} \cdot N \tag{4}$$

SW = story weight [Newton]

SF_L = scale factor length

SF_T = scale factor time

γ = typical weight of story, a value of $\gamma = 0.009$ [N/mm2] was used

N = number of stories

Using these assumptions, Eq. 1 can be rewritten as follows:

$$Sd = Sa \cdot \frac{M}{K} = Sa \cdot \frac{\alpha}{100} \cdot \frac{A_{fl}}{A_{ce} + A_{wt}} \qquad \text{or} \tag{5}$$

$$Sd = \frac{Sa \cdot \alpha}{SI} \qquad \text{where} \qquad SI = 100 \cdot \frac{A_{ce} + A_{wt}}{A_{fl}} \tag{6}$$

α = constant of proportionality

SI = structural index

A_{ce} = column area at the base

A_{wt} = wall area at the base

A_{fl} = total floor area of the structure

Substituting the spectral acceleration Sa using the expression for the linear response spectrum proposed by Shimazaki and Sozen (1984) of Sa = PGA*Aa into Eq. 6, the spectral displacement, Sd, can be rewritten as follows:

$$Sd = \frac{PGA \cdot Aa \cdot \alpha}{SI} \qquad \text{for } T < Tg \qquad (7)$$

$$Sd = \frac{PGA \cdot Aa \cdot \alpha}{SI} \cdot \frac{T}{Tg} \qquad \text{for } T > Tg \qquad (8)$$

PGA = peak ground acceleration

Aa = acceleration amplification factor; a value of 3.75 for systems with 2% damping factor is representative of a wide range of earthquakes (Shibata 1976)

T = period of the structure

Tg = characteristic period of the ground motion, it may be defined as the period at which the assumed constant acceleration region ends;
for practical applications, Lepage (1997) proposes to take the characteristic period equal to 0.6 sec for stiff and 1.2 sec for soft soil.

In Eq. 7, the numerator is independent of the structural properties and dependent only on properties of the ground motion, quantities that cannot be influenced by the designer.

Eq. 7 indicates that drift is a function of the structural index, SI, the peak ground acceleration, the amplification factor, and the ratio of the period of the structure to the characteristic period of the site. So far, the value for α is not known, but it will be investigated using the experimental results that were compiled.

Because it is not easy to relate the spectral displacement Sd to damage, Eq. 7 must be modified to obtain a relationship between SI and the mean drift ratio.

Combining the product in the numerator to a constant, k_1, and expressing the structural displacement, Sd in terms of the roof drift, S_{roof},

$$Sd = \frac{S_{roof}}{MPF} \tag{9}$$

MPF = modal participation factor; can be approximated by 1.3 for regular low rise frames (first mode)

Eq. 7 can be further transformed to

$$SI = \frac{k_1}{\frac{S_{roof}}{1.3}} = \frac{k_2}{S_{roof}} \quad \text{with} \quad k_1 = PGA \cdot Aa \cdot \alpha \quad \text{and} \quad k_2 = 1.3 \cdot k_1 \tag{10}$$

As indicated previously, α is a constant of proportionality.

With the exception of α, all quantities in Eq. 10 are known or can be determined by evaluation earthquake records. In order to find values for α, the results of earthquake simulator tests will be used.

Typical experimental studies with earthquake simulators are based on ground motions from a particular earthquake record, and the models are shaken to increasing levels of peak ground acceleration. Browning et al. (2000) looked at results from numerous earthquake simulator tests. A linear regression between drift and peak acceleration was carried out in order to normalize the response with respect to the spectral acceleration. This approximation was found to be adequate up to a performance level near collapse.

The maximum roof drift for a peak acceleration of 0.5g was calculated using the regression formulas mentioned before and was plotted with respect to 1/SI. A limit of 20/SI was established as a conservative estimate for the maximum roof drift in millimeters for a given acceleration of 0.5g (see Fig. 1). Using this limit, Eq. 10 can be transformed to

$$S_{roof} = k_2 \cdot \frac{1}{SI} \qquad \text{with} \qquad k_2 = 20 \tag{11}$$

Using the definitions for k_1 and k_2 this leads to

$$k_1 = \frac{k_2}{1.3} = \frac{20}{1.3} = PGA \cdot Aa \cdot \alpha \tag{12}$$

Based on the analysis by Browning et al. (2000), a value for α can be calculated for a peak ground acceleration of 0.5g and an amplification factor Aa of 3.75 as proposed by Shibata (1976):

$$\alpha = \frac{k_1}{PGA \cdot Aa} \tag{13}$$

$$\alpha = \frac{20}{1.3} \cdot \frac{1}{3.75} \cdot \frac{1}{0.5g} \tag{14}$$

Using the value for α shown in Eq. 14, a conservative estimate for the maximum spectral displacement in millimeters, given a peak ground acceleration of 0.5g, can be determined from Eq. 10 as follows:

$$SI = \frac{k_1}{Sd} = \frac{20}{1.3} \cdot \frac{1}{Sd} \qquad \Rightarrow \qquad Sd \approx \frac{15}{SI} \tag{15}$$

In order to obtain a dimensionless quantity that can be used to evaluate the performance of a structure, the mean drift ratio, MDR, is introduced:

$$MDR = \frac{S_{roof}}{N \cdot H} \tag{16}$$

N = number of stories

H = story height

Substituting Eq. 11 in Eq. 16 leads to:

$$MDR = \frac{20}{N \cdot H} \cdot \frac{1}{SI} \tag{17}$$

For a two-story structure having a typical story height of 2800mm, Eq. 17 becomes

$$MDR = \frac{0.4}{100} \cdot \frac{1}{SI} \qquad \text{or} \qquad SI = \frac{0.4}{100} \cdot \frac{1}{MDR} \tag{18}$$

Eq. 18 was derived for PGA of 0.5g. Since SI is proportional to the peak ground acceleration PGA, the same procedure can be used for peak ground accelerations that differ from 0.5g. For a peak ground acceleration of 0.75g for example, the value for SI would be adjusted by multiplying it by a factor of 0.75/0.5.

Eq. 18 is formulated for arbitrary peak ground acceleration,

$$SI = \frac{0.4}{100} \cdot \frac{1}{MDR} \cdot \frac{PGA}{0.5} = 0.008 \cdot \frac{PGA}{MDR} \tag{19}$$

where PGA is the peak ground acceleration divided by the acceleration of gravity.

2.1.2 Performance constant

To control the performance of the structure, a limit value for SI will be established that relates the seismic demand and the tolerable level of damage associated with the maximum drift to an acceptable level of performance. Cp is introduced as a limit value for SI to obtain a given level of performance.

Hassan and Sozen (1997) suggested Cp equal to 0.25 and 0.5 for performance levels of life safety and immediate occupancy respectively. These values were based on damage surveys obtained from buildings of one to five stories after the 1992 Erzincan earthquake in Turkey (PGA of approximately 0.5g). It is reasonable to assume that these values are conservative because the materials and details used in that area did not meet standards required by modern design codes, resulting in a more severe level of damage than would be expected in regions with stricter quality control.

Another approach is to determine values for Cp based on earthquake simulator tests.

Eq. 19 can be reformulated to

$$Cp = \frac{0.008 \cdot PGA}{MDR} \tag{20}$$

The following limits are set for a peak ground acceleration of 0.5g based on Eq. 20.

A mean drift ratio of 0.75% is considered as an appropriate limit for the performance level of immediate occupancy which leads to a value of Cp = 0.53. In the case of life safety, an allowable mean drift ratio of 2% results in Cp = 0.2. Both values are comparable to those proposed by Hassan and Sozen (1997).

The values for Cp can now be compared to the structural index, SI, calculated for the structure in question using Eq. $(6)_2$. If the structural index SI is larger than Cp, the structure can be considered appropriate for the given demand, if SI is smaller than Cp, the structure does not meet the requirements for the mean drift ratio chosen to be acceptable and the member sections must be adjusted.

2.1.3 Correction for structures with long periods:

For structures having a period T greater than the characteristic period of the ground motion T_g, the reduction of earthquake forces must be accounted for by including a reduction factor C_{tg}. For these structures, Eq. 8 will be modified. A simple approach is to assume that the period is proportional to the number of stories N. The characteristic period of the ground motion T_g can be associated with a limit number of stories, NT_g, resulting in a correction factor of the form $C_{tg} = N/NT_g$. Sozen and Browning (1997) proposed a correction factor C_{tg} equal to N/4 for structures with more than four stories.

This leads to the following design expressions:

$$Cp < SI \qquad \text{for} \qquad N < 4 \tag{21}$$

$$Cp < SI * N/4 \qquad \text{for} \qquad N > 4$$

2.1.4 Actual calculations for small scale structures

The flat-rate method requires calculating the ratio of the area of the structural elements to the total floor area. Because the structures for the earthquake simulator tests were scale-reduced models, an equivalent floor area was calculated based on the story weights according to Eq. 4. The definition of SI by Hassan and Sozen (1997) was adopted, and the structural index was calculated using Eq. $(6)_2$. The calculated structural index SI and the peak ground acceleration from each test were used to calculate the mean drift ratio, MDR, according to Eq. 22, which was derived from Eq. 18.

$$MDR = 0.008 \cdot \frac{PGA}{SI} \tag{22}$$

$$SI = (CI + WI) \cdot C_{tg}$$

MDR = mean drift ratio [-]

PGA = peak ground acceleration [g]

SI = structural index [-]

In the case of the structures tested by Oliva et al. (1987), Oliva (1980) and Hidalgo (1975), the length of the members was scaled by 7/10, but the weight was not scaled, so a typical weight of $\gamma = 0.009 * (SF_L)^2$ was used for the calculation of the equivalent floor area.

2.2 Target period method

While the flat rate method gives a coarse approximation of the period, the target period method can be used if a more accurate analysis is needed. The target period method uses a concept similar to the flat-rate method because the period of a given structure is compared to a limiting value. In this case the period is calculated using gross section properties of the elements, resulting in a more accurate estimate.

The method is still simple because it only requires the calculation of the initial period of the structure. To apply the method, the target period of each structure has to be determined. This is done based on the estimated earthquake demand at the site and the mass and stiffness distribution defined for the structure.

Earthquake demand is represented as a linearly increasing relationship between drift demand and the period of the structure. The displacement demand curve can easily be derived by modifying the displacement response spectrum for a given site using 2% of critical damping. Shimazaki (1984) concluded that a reasonable estimate to the upper bound for the expected drift could be obtained by using the simplified displacement response spectrum with a period equal to the initial period, Ti, multiplied by $\sqrt{2}$ (see Fig. 2).

Adequate performance can be assured if an allowable drift is not exceeded, and if the initial period of the structure is kept below the initial period associated with that drift.

A simple relationship can be used to determine the expected maximum drift, D:

$$D = \sqrt{2} \cdot T_i \cdot k \cdot MPF \tag{23}$$

D = expected maximum drift [mm]

k = slope of the demand curve (see Fig. 2)

MPF = modal participation factor for the first mode

T_i = initial period of the structure

The slope k depends on the earthquake demands at the site and can be estimated with the following equation (Lepage, 1997):

$$k = \frac{F_a \cdot T_g \cdot a}{(2\pi)^2} \tag{24}$$

F_a = amplification factor for regions of nearly constant acceleration

T_g = characteristic period of the design ground motion

a = design peak ground acceleration

According to Newmark et al. (1973), a value of 15/4 for F_a is appropriate for a wide range of site conditions. The characteristic period listed for each input ground motion was determined by comparing the relative shapes of energy response spectra and acceleration response spectra calculated using the proposed ground motion at 2% and 10% of critical damping (Lepage, 1997). For the design case, the specified corner period of the design ground motion would be appropriate.

Chapter 3

3.1 Properties of specimens

In this chapter, data describing the test structures in detail and the calculations for flat-rate and target period method are presented.

Table 1 contains general information about the specimens. They were built in reduced scale and small bars and scaled aggregates were used. The number of stories ranged from 1 to 10. Bay dimensions were 305mm by 914mm for the smallest test structure and 6000mm by 6000mm for the largest. The story weight was around 70 kN for a large number of models and the story height was between 550mm and 4570mm. The cross sectional dimensions of beams and columns ranged between 38mm by 38mm for beams and 51mm by 38mm for columns in the smallest model and 500mm by 300mm for beams and 450mm by 450mm for columns in the largest model.

Table 2 gives information used for the calculation of the structural index. It includes column and wall area per floor, the scale factors for length, which ranged between 1:1 and 1:12, and scale factors for time, which ranged between 0.426 and 1. The compressive strength of the concrete, f'_c, was around 30 N/mm^2 and the modulus of elasticity, E, was about 25000 N/mm^2 for most of the models.

3.2 Test input

Table 3 contains information about the actual earthquake simulator tests. Most of the shaking table excitations were based on the 1940 El Centro record, but records from Taft, Pacoima, Miyagi-ken-oki and Tokachi-oki also were used. These records were scaled for the tests. The minimum peak ground acceleration used was 0.07g, the maximum used was 3.4g. The maximum measured roof drifts and the natural frequencies of the structures also are listed.

3.3 Calculations for the drift

The results obtained using the flat-rate method are presented in Tables 4 and 5. Table 6 contains theses quantities for the structures previously evaluated by Browning et al. (2000). The structural index varied between 0.01 for the softest structure and 13.93 for the stiffest structure. A structure of this extreme stiffness is not likely to be built in reality though, since the cost would be high. The ratios of calculated to measured mean drift ratio ranged between 0.033 for the stiffest and 6.79 for the softest structure, but the majority of tests lead to a ratio of approximately 1.

The results obtained using the target period method are shown in Tables 7 and 8. In this case, the ratios between calculated and measured drift ranged between 0.77 and 7.43.

Table 9 summarizes the average values and the standard deviation for mean drift ratio and estimated drift and shows the maximum and minimum peak ground accelerations per test structure.

3.4 Charts

In order to show the relation between the stiffness of the structure and the calculated results, the ratio of calculated to measured MDR is plotted against the structural index, SI, and against the ratio of girder length to girder depth (Figs. 3 through 6). In Figs. 3 and 5, SI and MDR were calculated using the original equations proposed by Hassan and Sozen, in Figs. 4 and 6, the adjusted equation to calculate WI was used. In the adjusted equation, a factor of 0.5 is included to calculate the wall index, WI, equivalent to the calculation of the column index, CI:

$$WI = \frac{A_{wt} \cdot 100}{A_{eq} \cdot N} \cdot 0.5 \qquad (24)$$

Figs. 3 and 4 show that the ratio of calculated to measured mean drift ratio is decreasing with increasing stiffness, characterized by an increasing structural index. Figs. 3 and 4 only differ for the structures with 6 to 10 stories because the 1 to 3 story models investigated had no walls and are therefore not influenced by a changed wall index.

Figs. 5 and 6 show the same trend, indicating lower ratios of calculated to measured MDR, which means a closer approximation of the experimentally obtained response, for stiff structures, characterized by lower values of L/h. Very soft structures with L/h ratios larger than 20 show a reversed trend. For the same reasons as mentioned before, Figs. 5 and 6 only differ for structures that have 6 to 10 stories.

Figs. 7 and 8 present the MDR ratios plotted versus the peak ground acceleration, once with the original equation for WI, once with the above mentioned adjusted equation. In these charts, no trend can be observed which means that the quality of the calculated response seems to be independent of the peak ground acceleration.

Fig. 9 shows the relation of structural stiffness, here characterized by the initial period of the structure, Ti, to the ratio of calculated to measured drift obtained with the target period method. It can be observed that the ratio of calculated to measured drift is closer to 1, indicating a better approximation of the drift, for structures with a low initial period.

Fig. 10, shows the equivalent to Figs. 7 and 8, which means the ratio of calculated to measured drift is plotted versus the peak ground acceleration of each test. For the target period method, larger peak ground accelerations seem to lead to better approximations of the structural response which means a smaller ratio of calculated to measured drift.

Chapter 4

Results and Conclusions

A total of 160 experiments were considered in this study. The following can be said about the applicability of the flat-rate and the target period method:

Overall, the calculated drift obtained using either one of the methods matched the experimental results quite well although it was not always a conservative estimate.

4.1 Flat-rate method

4.1.1 Effect of stiffness

It can be seen that the mean drift ratio, MDR, was often underestimated, especially for stiff structures with a structural index, SI, higher than 0.22. For the stiffest structure considered, the drift was underestimated by 95% (Wolfgram, SI = 13.93; the results obtained from this model are not shown in the graphs because they are considered irrelevant for practical considerations). For soft structures with SI ranging between 0.01 and 0.18, the mean drift ratio was highly overestimated, for the softest structure in this study up to 400% (Eto et al., SI = 0.01).

4.1.2 Effect of number of stories

For the 1 to 3 story structures, the average of calculated to measured MDR was 1.68, the standard deviation was 1.34, for the 6 to 10 story structures, the average was

1.50 and the standard deviation was 0.88. This indicates that for the models considered, the number of stories had no direct influence on the quality of the calculated results.

4.1.3 Effect of walls

Two models that had walls incorporated in the structure, show that the equation to calculate the wall index, WI, proposed by Hassan and Sozen (1997) leads to an overestimation of the stiffness of the structure. The relatively high wall index calculated based on Hassan's and Sozen's suggested equation leads to a high value of SI which, as mentioned before, yields to an underestimation of the mean drift ratio. In a second set of calculations, the wall index was calculated using an equation equivalent to the equation for the column index, CI, (see Eq. 24), including the factor of 0.5. Applying this modified equation, a lower structural index was calculated for the structures in question and the match between calculated and measured mean drift ratio could be improved significantly. Although more data is necessary to validate this conclusion.

4.2 Target period method

4.2.1 Effect of stiffness

As can be observed in Fig. 9, small initial periods, Ti, resulted in most cases in a better estimate of the drift.

4.2.2 Effect of number of stories and peak ground acceleration

For the target period method, slightly better results were obtained for taller structures. For 1 to 3 story structures, the average of calculated to measured drift was 2.42, the standard deviation was 1.50, for 6 to 10 story structures, the average was 1.58, the standard deviation was 0.93.

According to Fig. 10, a higher peak ground acceleration lead to slightly better results, but this trend was not always consistent.

For the wall frame structures tested by Vintzilieou (1998) and by Moehle and Sozen (1980), the following could be observed: In the case of these structures, a higher peak ground acceleration resulted in a lower ratio of calculated to measured drift, which in the case of Vintzileou improved the quality of the results, in the case of Moehle et al. lead to an underestimation of the maximum drift. This observation is also true for a large number of Hidalgo's and Kitagawa's and Eto's tests.

4.3 Flat rate method versus target period method

4.3.1 Separate consideration of over- and underestimation

Comparing the quality of the results of flat-rate versus target period method, the target period method gives more accurate results if over- and underestimation are considered separately. While the flat-rate method underestimated the mean drift ratio for 30% of the tests, the target period method underestimated the drift for only 10%. Also, the average underestimation of the drift of 13% for the target period method was much better than the average underestimation for the flat-rate method of 37%. Although the target period method tended to overestimate the drift, which was the case in 141 tests, the average of overestimation of the drift was only 88% compared to 109% overestimation of the mean drift ratio for the flat-rate method.

4.3.2 Combined consideration of over- and underestimation

Considering all results, over- and underestimated combined, the flat-rate method seems to give a better estimate of the drift of the structures. Summarizing this, it can be said that the range of over- or underestimation of the drift is smaller for the target period method and that this method is also more conservative, but that the overall estimate of the drift is better for the flat-rate method. As can be seen in Table 9, the average of calculated to measured MDR for the 1 to 3 story buildings using the flat-rate method was 1.68, the standard deviation was 1.14, compared to a ratio of calculated to measured drift of 2.42 and a standard deviation of 1.5 for the target period method. This indicates that the flat-rate method should be used especially for low-rise structures.

No trend could be observed when the drift was underestimated or highly overestimated by the target period method. It could be seen though that the target period method underestimated the drift in most cases for the same models and tests for which the flat-rate method had underestimated the mean drift ratio (Hidalgo, test W7 and W9, Moehle, FHW and FFW for the higher peak ground acceleration, and Wolfgram, last tests of NS2 and NS3) and that the percentage of underestimation of the mean drift ratio for these tests was the ‘highest’ within the set of tests of the structure.

		Bay Dimensions						Beam Dim. mm		Column Dim. mm		Wall Dim. mm			
Structure / Report	Number of Bays	L (mm)	B (mm)	Number of Stories	Level	Story Height, mm	Story Weight, kN	b	h	b	h	t	l	Slab Thickness (mm)	No. of Columns per Floor (1st Fl)
1) Oliva, Clough	1 * 1	3657.6	914.4	2	1	2316.48	71.17	146.05	288.93	215.9	146.05	-	-	73.03	4
1987					2		35.58	146.05	288.93	215.9	146.05	-	-		4
Coupled Frame															
(1 Fr.)															
2) Oliva	1 * 1	3663.75	914.4	2	1	2009.14	71.17	146.05	288.93	215.9	146.05	-	-	73.025	4
1980					2	2009.14	35.59	146.05	288.93	215.9	146.05	-	-		4
Coupled Frame															
3) Hidalgo	1 * 1	3657.6	914.4	2	1	2009.14	71.17	146.05	288.93	215.9	146.05	-	-	73.025	4
1975					2	2009.14	35.59	146.05	288.93	215.9	146.05	-	-		4
Coupled Frame															
5) Kitagawa	1 * 1	3000	1000	2	1	1875	11.1	300	150	200	200	-	-	60	4
1984				-	2	1500		300	150	200	200	-	-		4
Coupled Frames				-	base	-		500	200	-	-	-	-		-
(2 ident. Fr.)															
6) Eto, Hiroaki, Takeda				3		560		70	140	120	120	-	-	NA	1
1980															
Frames YD1	2 * 0	1100	0	-			X*								
YD2	2 * 0	1100	0	-			88.26								
YD2	2 * 0	1100	0												
(2 ident. Fr.)															

Table 1: General dimensions of the test structures

		Bay Dimensions						Beam Dim. mm '		Column Dim. mm '		Wall Dim. mm '			
Structure / Report	Number of Bays	L (mm)	B (mm)	Number of Stories	Level	Story Height, mm	Story Weight, kN	b	h	b	h	t	l	Slab Thickness (mm)	No. of Columns per Floor (1st Fl)
7) Minowams, Ohtani, Ogawa: 1994 Coupled Frames															
Type A	1 * 1	6000	6000	1	1*	2000	X*	500	300	350	350	-	-	150	4
						-	1010			-	-	-	-		
Type B	1 * 1	6000	6000	1				500	300					150	
Type C	1 * 1	6000	6000	3	1-3	3000	294	500	300	450	450	-	-	150	4
					roof			400	250	-	-	-	-		
9) Vintzileou, Yong Lu, Zhang 1998															
Frame 1 (BF1)	3 * 0	2370	0	6	1	910	XX*	90.91	72.73	109.09	109.09	-	-	not	4
(dim. ident. for					2	550	67.79	90.91	72.73	90.91	90.91	-	-	given	4
BF1 and SWF)					3,4	550		81.82	72.73	81.82	81.82	-	-		4
					5,6	550		72.73	63.64	63.64	63.64	-	-		4
Frame 4 (SWF)	3 * 0	2370	0	6	1	910	XX*	81.82	72.73	81.82	81.82	50	550	not	4
wall-frame syst.					2-6	550	72.40	81.82	72.73	81.82	81.82			given	4
w/ centralised															
structural wall															
10) Moehle, Sozen 1980	3 * 1	305	914	9	1	4570	4.45	38	38	51	38	38	203	NA	8
Note A					2-9	2290	4.45								
Struct. 1 FNW															
Struct. 2 FSW															
Struct. 3 FHW															
Struct. 4 FFW															

X* : load is applied at the top of the structure
XX* : total added mass
XXX* : tests, not different frames

Table 1 cont.: General dimensions of the test structures

Note A: Abbreviations

FNW: Frames with no wall
FSW: Frames with one-story ("stub") wall
FHW: Frames with four-story ("half") wall
FFW: Frames with nine-story ("full-height") wall

Note B: Complete peak ground accelerations and top displacements for frames 1 and 4 (Vintzileou)

Typical Testing Sequence:		Top Displacements (mm):			
		BF1		SWF	
		pos. dir.	neg. dir.	pos. dir.	neg. dir.
1. Random-Vibration Test					
2. Earthquake Simulation Test	EL 0.10	6.14	7.50	1.14	2.27
3. Random-Vibration Test					
4. Earthquake Simulation Test	EL 0.30	22	27.0	6.82	10.23
5. Random-Vibration Test					
6. Earthquake Simulation Test	EL 0.60	44	34.0	20.68	22.73
7. Random-Vibration Test					
8. Earthquake Simulation Test	EL 0.90	61	51.5	46.59	40.45 *
9. Random-Vibration Test					
10. Earthquake Simulation Test	EL 1.20	82	88.6	--	69.32

EL** ~ El Centro, PGA = ** g
* rupture of wall reinforcement

			Crossectional areas				Scale Factors		Material Properties	
Structure / Report	Level	No. of Columns per Floor (1st Fl)	Column Area per Floor, [mm^2]	No. of Walls per Floor	Wall Area per Floor, [mm^2]	Total height of specimen [mm]	ScaleFactor for Length, SL	ScaleFactor for Time, ST	f'c [N/mm^2]	Modulus of Elasticity, Ec [N/mm^2]
1) Oliva, Clough	1	4	126128.78	-	-	4632.96	7/10	1	32.0	19326
1987	2	4	126128.78	-	-					
Coupled Frame										
(1 Fr.)										
2) Oliva	1	4	126128.78	-	-	4425.95	7/10	1	27.58	24840
1980	2	4	126128.78	-	-					*
Coupled Frame									32.0	26757
									test date	*
3) Hidalgo	1	4	126128.78	-	-	4425.95	7/10	1	27.58	24840
1975	2	4	126128.78	-	-					*
Coupled Frame										
5) Kitagawa	1	4	160000	-	-	3375	1/2	1/2	24	23172
1984	2	4	160000	-	-					*
Coupled Frames		-		-	-					
(2 ident. Fr.)										
6) Eto, Hiroaki, Takeda										
1980										
Frames YD1	1-3	1	14400	-	-	1690	1/6	1	32.36	21575
YD2	1-3	1	14400	-	-	1690	1/6	1	32.36	21575
(2 ident. Fr.)										
7) Minowams, Ohtani, Ogawa: 1994 Coupled Frames Type A	1	4	490000	-	-	4162	1/1	1	35.10	28023.00
Type B									35.00	27983.06

*: E calculated based on ACI: E = 4730 * sqrt(f'c)

Table 2: Cross sectional areas, scale factors and material properties

		Crossectional areas					Scale Factors		Material Properties	
Structure / Report		No. of Columns per Floor (1st Fl)	Column Area per Floor, [mm^2]	No. of Walls per Floor	Wall Area per Floor, [mm^2]	Total height of specimen [mm]	ScaleFactor for Length, SL	ScaleFactor for Time, ST	f'c [N/mm^2]	Modulus of Elasticity, Ec [N/mm^2]
9) Vintzileou, Yong Lu, Zhang 1998										
Frame 1 (BF1)	1	4	47603.31	-	-	3660	1/5.5	0.426	30	25907
(dim. ident. for	2	4	33057.85	-	-					*
BF1 and SWF)	3,4	4	26776.86	-	-					
	5,6	4	16198.35	-	-					
Frame 4 (SWF)	1	4	26776.86	1	27500	3660	1/5.5	0.426	30	25907
wall-frame syst.	2-6	4	26776.86	1						*
w/ centralised										
structural wall										
10) Moehle, Sozen 1980	1	8	15504	1	7714	2286	1/12	0.4	35 - 40	27983 -
Note A	2-9		15504							29915
Struct. 1 FNW										
Struct. 2 FSW										
Struct. 3 FHW										
Struct. 4 FFW										

* E calculated based on ACI: E = 4730 * sqrt(f'c)

Table 2 cont.: Cross sectional areas, scale factors and material properties

Structure / Report	Earthquake	Comments / Test	Peak Base Acceleration, [g]	Frame	Natural Frequency [sec.] measured	Natural Frequency [sec.] calculated	Comments (calculated/measured)	Max. Measured Roof Drift [mm]
1) Oliva, Clough	Taft N 69W	biax-weak ax	0.306		Note 1	0.28		39.12
1987		biax-strong ax	0.684			**		53.85
Coupled Frame		uniaxial	0.798					51.82
(1 Fr.)								only 1st floor given
2) Oliva	Taft N69W	Taft100 (1)	0.062	RC5		0.23		12.78
1980	Taft N69W	Taft100 (2)	0.061	biax. Test		**		12.65
Coupled Frame	Taft N69W	Taft1000	0.685					128.78
	Pacoima	Pacoima	1.49					132.84
	Pacoima	Pacoima, repaired frame	1.37					133.35
	Taft N69W	Taft1000, repaired frame	0.711					128.52
3) Hidalgo	Taft	without concrete blocks	0.07	XXX* N1	-	0.23		2.87
1975	Taft	without lateral bracing	0.11	N2		**		5.44
Coupled Frame	Taft		0.22	N3				14.48
	Taft		0.24	N4				18.57
	Taft	with concrete blocks	0.07	W1				8.71
	Taft	with lateral bracing	0.1	W2				21.77
	Taft		0.22	W3				38.74
	Taft		–	W4				–
	Taft		0.3	W5				38.74
	Taft		0.44	W6				73.71
	Taft		0.3	W7				86.33
	Taft		0.22	W8				55.85
	El Centro		0.15	W9				40.69
	El Centro		0.46	W10				92.51
	Taft	repaired, with concrete	0.08	R1				8.18
	Taft	blocks, with lateral bracing	0.41	R2				75.06
	Taft		0.64	R3				113.84
				XXX*				

** calculated using Sarsan

Table 3: Peak ground accelerations, max. roof drift and natural frequencies of the test structures

Structure / Report	Earthquake	Comments / Test	Peak Base Acceleration, [g]	Frame	Natural Frequency [sec.] measured	Natural Frequency [sec.] calculated	Comments (calculated/measured)	Max. Measured Roof Drift [mm]
5) Kitagawa 1984 Coupled Frames (2 ident. Fr.)	N-S Miyagi-ken-oki		0.216 0.566	DR 10 DR 20	0.33 0.46	- -		12.3 50.1
6) Eto, Hiroaki, Takeda 1980 Frames YD1	NS Tokachi-Oki		0.271	YD1	0.29	0.24		95.9
YD2	Taft S69E	1st run	0.197	YD2	0.29	0.24		32.0
YD2 (2 ident. Fr.)	Taft S69E	2nd run	0.308	YD2	0.29	0.24		scale out
7) Minowams, Ohtani, Ogawa: 1994 Coupled Frames Type A	EW Tokachi-Oki	1st excitat.	0.816	Type A	0.24	-	measured before testing	144
		2nd excitat.	0.744	Type A	0.31	-	measured after testing	165
Type B	EW Tokachi-Oki	one exc.only	0.642	Type B	0.23	-	measured before testing	109

** calculated using Sarsan

Table 3 cont.: Peak ground accelerations, max. roof drift and natural frequencies of the test structures

Structure / Report	Earthquake	Comments / Test	Peak Base Acceleration, [g]	Frame	Natural Frequency [sec.] measured	Natural Frequency [sec.] calculated	Comments (calculated/measured)	Max. Measured Roof Drift [mm]
9) Vintzileou, Yong Lu, Zhang 1998								
Frame 1 (BF1)	NS of El Centro	--	0.1	BF1	-	0.41		7.5
dim. ident. for			0.3	BF1		**		27.0
BF1 and SWF)			0.6	BF1				44.3
			0.9	BF1				60.7
			1.2	BF1				88.6
								Note B
Frame 4 (SWF)	NS of El Centro	--	0.1	SWF	-	0.42		2.27
wall-frame syst.			0.3	SWF		**		10.23
w/ centralised			0.6	SWF				22.73
structural wall			0.9	SWF			rupture of wall reinf.	46.59
			1.2	SWF				69.32
10) Moehle, Sozen 1980	NS of El Centro	–	0.4					
Note A			1st / 2nd acc.:					1st / 2nd acc.:
Struct. 1 FNW			0.39 / 0.78	FNW		-		19.5 / 44
Struct. 2 FSW			0.34 / 0.59	FSW		-		18.2 / 40
Struct. 3 FHW			0.41 / 0.48	FHW		-		17.6 / 40
Struct. 4 FFW			0.32 / 0.55	FFW		-		17.3 / 44

** calculated using Sarsan

Table 3 cont.: Peak ground accelerations, max. roof drift and natural frequencies of the test structures

Structure	Stories	averaged weight used for calculations	ScaleFactor for Length, SL	ScaleFactor for Time, ST	Typical Weight [N/mm^2]	Equivalent Floor Area [mm^2]	Column Area per Floor [mm^2]	Number of Storeys	Column Index CI	Wall Area per Floor, mm^2	Wall Index WI	Correction Factor	Structural Index SI
1) Oliva,	2	53.38	0.7	1	0.00441	17291869.13	126129	2	0.18	-	-	1	0.18
Clough					**		126129						0.18
													0.18
2) Oliva	2	53.38	0.7	1	0.00441	17291435.05	126129	2	0.18	-	-	1	0.18
					**		126129						0.18
													0.18
													0.18
													0.18
													0.18
3) Hidalgo	2	53.38	0.7	1	0.00441	17291184	126129	2	0.18	-	-	1	0.18
					**		126129			-	-		0.18
													0.18
													0.18
													0.18
													0.18
													0.18
													0.18
													0.18
													0.18
													0.18
													0.18
													0.18
													0.18
													0.18
4) Kitagawa	2	11.1	0.5	2	0.009	9866667	160000	2	0	-	-	1	0.41
							160000	-		-	-		0.41
5) Eto, Hiroaki, Takeda	2	29.42	0.1667	1	0.009	19609411.45	14400	3	0.01	-	-	1	0.01
YD1								-					0.01
YD2								-					0.01
YD2													

** the typical weight was adjusted in these cases by a factor of (0.7^2), because the length of the models was scaled, SL = 0.7, but the weight was not scaled.

Table 4: Calculation of column index, CI, wall index, WI, and structural index, SI.

Structure	Stories	averaged weight used for calculations	ScaleFactor for Length, Sl	ScaleFactor for Time, St	Typical Weight [N/mm^2]	Equivalent Floor Area [mm^2]	Column Area per Floor [mm^2]	Number of Storeys	Column Index CI	Wall Area per Floor [mm^2]	Wall Index WI	Correction Factor	Structural Index SI
6) Minowams, Ohtani, Ogawa Type A	1	1010.09	1	1	0.009	112232222.2	490000	1	0.22	-	-	1	0.22
								-		-	-		0.22
Type B		1010.09							0.22			1	0.22
7) Vintzileou, Yong Lu, Zhang													
Frame 1 (BF1)	6	11.30	0.182	0.426 [=1/sqrt(5.5)]	0.009	1255370	47603.31	6	0.32	-	-	1.5	0.47
			[=1/5.5]							-	-		0.47
										-	-		0.47
										-	-		0.47
													0.47
Frame 4 (SWF)	6	12.07	0.182	0.426 [=1/sqrt(5.5)]	0.009	1340741	26776.86	6	0.17	27500	0.34	1.5	0.76
			[=1/5.5]										0.76
													0.76
													0.76
													0.76
8) Moehle, Sozen Structure 1 FNW	1	4.45	0.08333	0.4	0.009	949333	15504	9	0.09	7714	0.09	2.25	0.41
			[=1/12]	[=1/2.5]				no wall					0.41
													0.41
Structure 2 FSW								wall in storey 1					0.41
													0.41
Structure 3 FHW								w in st. 1-4					0.41
													0.41
Structure 4 FFW								w in st.1-9					0.41
													0.41

Table 4 cont.: Calculation of column index, CI, wall index, WI, and structural index, SI.

Wall index, WI, calculated including a factor of '1/2'

Structure	Stories	averaged weight used for calculations	ScaleFactor for Length, Sl	ScaleFactor for Time, St	Typical Weight [N/mm^2]	Equivalent Floor Area [mm^2]	Column Area per Floor [mm^2]	Number of Storeys	Column Index CI	Wall Area per Floor [mm^2]	Wall Index WI	Correction Factor	Structural Index SI
7) Vintzileou, Yong Lu, Zhang Frame 4 (SWF)	6	12.0667	0.182 [=1/5.5]	0.426 [=1/sqrt(5.5)]	0.009	1340741	26776.86	6	0.17	27500	0.17	1.5	0.51
													0.51
													0.51
													0.51
													0.51
8) Moehle, Sozen	1	4.45	0.08333 [=1/12]	0.4 [=1/2.5]	0.009	949333	15504	9	0.09	7714	0.05	2.25	0.31
Structure 1 FNW								no wall					0.31
													0.31
Structure 2 FSW								wall in storey 1					0.31
													0.31
Structure 3 FHW								w in st. 1-4					0.31
													0.31
Structure 4 FFW								w in st. 1-9					0.31
													0.31

Table 4 cont.: Calculation of column index, CI, wall index, WI, and structural index, SI.

Structure	Peak acceleration [g] (1st acc.)	Max. Measured Roof Drift [mm] (1st acc.)	MDR (Mean drift ratio) calculated	MDR (Mean drift ratio) measured	MDR calc / MDR measured	Girder length, L (mm)	Girder height, h [mm]	L / h	Average MDRc/MDRm ratio per test structure	Standard deviation
1) Oliva,	0.306	39.12	0.0134	0.0084	1.59	3657.6	288.93	12.66		
Clough	0.684	53.85	0.0300	0.0116	2.58	3657.6	288.93	12.66		
	0.798	51.82	0.0350	0.0112	3.13	3657.6	288.93	12.66	2.43	0.781
2) Oliva	0.062	12.78	0.0027	0.0029	0.94	3663.75	288.93	12.68		
	0.061	12.65	0.0027	0.0029	0.94	3663.75	288.93	12.68		
	0.685	128.78	0.0301	0.0291	1.03	3663.75	288.93	12.68		
	1.49	132.84	0.0654	0.0300	2.18	3663.75	288.93	12.68		
	1.37	133.35	0.0601	0.0301	1.99	3663.75	288.93	12.68		
	0.711	128.52	0.0312	0.0290	1.07	3663.75	288.93	12.68	1.36	0.568
3) Hidalgo	0.07	2.87	0.0031	0.0006	4.74	3657.6	288.93	12.66		
	0.11	5.44	0.0048	0.0012	3.93	3657.6	288.93	12.66		
	0.22	14.48	0.0097	0.0033	2.95	3657.6	288.93	12.66		
	0.24	18.57	0.0105	0.0042	2.51	3657.6	288.93	12.66		
	0.07	8.71	0.0031	0.0020	1.56	3657.6	288.93	12.66		
	0.10	21.77	0.0044	0.0049	0.89	3657.6	288.93	12.66		
	0.22	38.74	0.0097	0.0088	1.10	3657.6	288.93	12.66		
	0.30	38.74	0.0132	0.0088	1.50	3657.6	288.93	12.66		
	0.44	73.71	0.0193	0.0167	1.16	3657.6	288.93	12.66		
*TPU	0.30	86.33	0.0132	0.0195	0.67	3657.6	288.93	12.66		
	0.22	55.85	0.0097	0.0126	0.76	3657.6	288.93	12.66		
*TPU	0.15	40.69	0.0066	0.0092	0.72	3657.6	288.93	12.66		
	0.46	92.51	0.0202	0.0209	0.97	3657.6	288.93	12.66		
	0.08	8.18	0.0035	0.0018	1.90	3657.6	288.93	12.66		
	0.41	75.06	0.0180	0.0170	1.06	3657.6	288.93	12.66		
	0.64	113.84	0.0281	0.0257	1.09	3657.6	288.93	12.66	1.72	1.211
4) Kitagawa	0.216	12.3	0.0043	0.0036	1.17	3000	150	20.00		
	0.566	50.1	0.0112	0.0148	0.75	3000	150	20.00	0.96	0.296
5) Eto, Hiroaki, Takeda										
YD1	0.271	95.9	0.1773	0.0567	3.12	1100	140	7.86		
YD2	0.197	32.0	0.1286	0.0189	6.79	1100	140	7.86	4.96	2.595
YD2	0.308	scale out								

*TPU : For these tests, the target period method underestimated the drift.

Table 5: Calculated mean drift ratio, MDR, and ratio of calculated to measured MDR and percentage of deviation.

Structure	Peak acceleration [g]	Max. Measured Roof Drift [mm]	MDR (Mean drift ratio) calculated	MDR (Mean drift ratio) measured	MDR calc / MDR measured	Girder length, L (mm)	Girder height, h [mm]	L / h	Average MDRc/MDRm ratio per test structure	Standard deviation
6) Minowams, Ohtani, Ogawa Type A	0.816	144	0.0299	0.0346	0.86	6000	300	20.00		
	0.744	165	0.0273	0.0396	0.69	6000	300	20.00		
Type B	0.642	109	0.0235	0.0262	0.90	6000	300	20.00	0.82	0.113
7) Vintzileou, Yong Lu, Zhang										
Frame 1 (BF1)	0.1	7.5	0.0017	0.0020	0.82	2370	72.73	32.59		
	0.3	27.0	0.0051	0.0074	0.69	2370	72.73	32.59		
	0.6	44.3	0.0101	0.0121	0.84	2370	72.73	32.59		
	0.9	60.7	0.0152	0.0166	0.92	2370	72.73	32.59		
	1.2	88.6	0.0203	0.0242	0.84	2370	72.73	32.59		
Frame 4 (SWF)	0.1	2.27	0.0010	0.0006	1.69	2370	72.73	32.59		
	0.3	10.23	0.0031	0.0028	1.13	2370	72.73	32.59		
	0.6	22.73	0.0063	0.0062	1.01	2370	72.73	32.59		
	0.9	46.59	0.0094	0.0127	0.74	2370	72.73	32.59		
	1.2	69.32	0.0126	0.0189	0.66	2370	72.73	32.59	0.93	0.302
8) Moehle, Sozen Structure 1 FNW	0.39	19.5	0.0077	0.0085	0.90	305	38	8.03		
	0.78	44.0	0.0153	0.0192	0.80	305	38	8.03		
Structure 2 FSW	0.34	18.2	0.0067	0.0080	0.84	305	38	8.03		
	0.59	40.0	0.0116	0.0175	0.66	305	38	8.03		
Structure 3 FHW	0.41	17.6	0.0081	0.0077	1.05	305	38	8.03		
*TPU	0.48	40.0	0.0094	0.0175	0.54	305	38	8.03		
Structure 4 FFW	0.32	17.3	0.0063	0.0076	0.83	305	38	8.03		
*TPU	0.55	44.0	0.0108	0.0192	0.56	305	38	8.03	0.77	0.173

*TPU: For these tests, the target period method underestimated the drift.

Table 5 cont.: Calculated mean drift ratio, MDR, and ratio of calculated to measured MDR and percentage of deviation.

Wall index,WI, calculated including a factor of '1/2'

Structure	Peak acceleration [g]	Max. Measured Roof Drift [mm]	MDR (Mean drift ratio) calculated	MDR (Mean drift ratio) measured	MDR calc / MDR measured	Girder length, L (mm)	Girder height, h [mm]	L / h	Deviation of calc. to meas. MDR in % (pos. values characterize overestimation of MDR)	Standard deviation
7) Vintzileou, Yong Lu, Zhang Frame 4 (SWF)	0.1	2.27	0.0016	0.0006	2.55	2370	72.73	32.59		
	0.3	10.23	0.0047	0.0028	1.70	2370	72.73	32.59		
	0.6	22.73	0.0095	0.0062	1.53	2370	72.73	32.59		
	0.9	46.59	0.0142	0.0127	1.12	2370	72.73	32.59		
	1.2	69.32	0.0190	0.0189	1.00	2370	72.73	32.59	1.58	0.574
8) Moehle, Sozen										
Structure 1 FNW	0.39	19.5	0.0102	0.0085	1.20	305	38	8.03		
	0.78	44.0	0.0201	0.0192	1.05	305	38	8.03		
Structure 2 FSW	0.34	16.2	0.0089	0.0080	1.12	305	38	8.03		
	0.59	40.0	0.0152	0.0175	0.87	305	38	8.03		
Structure 3 FHW	0.41	17.6	0.0107	0.0077	1.39	305	38	8.03		
*TPU	0.48	40.0	0.0124	0.0175	0.71	305	38	8.03		
Structure 4 FFW	0.32	17.3	0.0084	0.0076	1.11	305	38	8.03		
*TPU	0.55	44.0	0.0142	0.0192	0.74	305	38	8.03	1.02	0.235

*TPU: For these tests, the target period method underestimated the drift.

Table 5 cont.: Wall index, WI, calculated including a factor of "1/2", equivalent to the calculation of the column index, CI.

AVG: average deviation of MDR calc to MDR measured in percent
AVGgen: ~ for all data of one test structure
AVGu: ~ for underestimated values of one test structure
AVGo: ~ for overestimated values of one test structure

Structure	Stories	Correction Factor	Structural Index, SI	Girder length, L [mm]	Girder height, h [mm]	L / h	Peak acceleration [g] (1st acc.)	MDR (Mean drift ratio) **calculated**	MDR (Mean drift ratio) **measured**	MDR **calc** / MDR **measured**	Average MDRc/MDRm ratio per test structure	Standard deviation
Bracci et al. B1	3	1	0.08				0.2	0.020	0.0092	2.183		
			0.08				0.3	0.030	0.0163	1.838	2.01	0.244
Schulz 1985 SS1	10	2.5	0.1	610	57	10.7	0.35	0.0280	0.012	2.333		
			0.1	610	57	10.7	0.34	0.0272	0.010	2.720		
			0.1	610	57	10.7	0.53	0.0424	0.015	2.827		
SS2			0.1	610	57	10.7	0.35	0.0280	0.011	2.545		
			0.1	610	57	10.7	0.40	0.0320	0.013	2.462		
			0.1	610	57	10.7	0.36	0.0288	0.010	2.880		
			0.1	610	57	10.7	0.66	0.0528	0.018	2.933		
			0.1	610	57	10.7	0.99	0.0792	0.024	3.300		
			0.1	610	57	10.7	1.30	0.1040	0.040	2.600	2.73	0.291
Filiatraut '98 R2	2	1	0.11				0.21	0.0153	0.016	0.955		
			0.11				0.42	0.0305	0.032	0.964		
R4			0.11				0.21	0.0153	0.015	1.018		
			0.11				0.42	0.0305	0.024	1.273	1.05	0.149
Holiday Inn Van Nuys	7	1.8	0.11	5720	560	10.2	0.47	0.0342	0.012	2.848	2.84	-
Eberhard 1989 ES1	9	2.3	0.15	610	57	10.7	0.36	0.0192	0.009	2.233		
			0.15	610	57	10.7	0.52	0.0277	0.014	1.981		
			0.15	610	57	10.7	0.62	0.0331	0.020	1.653	*TPU	
ES2			0.15	610	57	10.7	0.35	0.0187	0.008	2.424		
			0.15	610	57	10.7	0.52	0.0277	0.015	1.849		
			0.15	610	57	10.7	0.61	0.0325	0.019	1.712	1.98	0.302
Healy 1978 MF1	10	2.5	0.15	305	38	8.03	0.40	0.0213	0.010	2.133		
			0.15	305	38	8.03	0.93	0.0496	0.021	2.362		
			0.15	305	38	8.03	1.40	0.0747	0.028	2.667	2.39	0.268
Moehle 1978 MF2	10	2.5	0.15	305	38	8.03	0.38	0.0203	0.010	2.027		
			0.15	305	38	8.03	0.83	0.0443	0.018	2.459		
			0.15	305	38	8.03	1.30	0.0693	0.024	2.889	2.46	0.431
Cecen 1979 H1	10	2.5	0.15	305	38	8.03	0.36	0.0192	0.013	1.477		
			0.15	305	38	8.03	0.84	0.0448	0.023	1.948		
			0.15	305	38	8.03	1.60	0.0853	0.044	1.939		

Table 6: Structural Index, ratios of calculated to measured mean drift ratio (flat rate method), L/h ratio

Structure	Stories	Correction Factor	Structural Index, SI	Girder length, L [mm]	Girder height, h [mm]	L / h	Peak acceleration [g] (1st acc.)	MDR (Mean drift ratio) calculated	MDR (Mean drift ratio) measured	MDR calc / MDR measured	Average MDRc/MDRm ratio per test structure	Standard deviation
Cecen 1979												
H2			0.15	305	38	8.03	0.17	0.0091	0.004	2.325		
			0.15	305	38	8.03	0.33	0.0176	0.008	2.228		
			0.15	305	38	8.03	0.49	0.0261	0.010	2.613		
			0.15	305	38	8.03	0.47	0.0251	0.011	2.279		
			0.15	305	38	8.03	0.72	0.0384	0.017	2.259		
			0.15	305	38	8.03	1.00	0.0533	0.025	2.133		
			0.15	305	38	8.03	2.60	0.1387	0.045	3.081	2.23	0.425
Wood 1986	10	2.5	0.17	610	57	10.7	0.44	0.0207	0.011	1.882		
Stepped			0.17	610	57	10.7	0.61	0.0287	0.014	2.050		
			0.17	610	57	10.7	0.90	0.0424	0.017	2.491	2.14	0.315
Aristizabal 1979												
D1	10	2.5	0.18	279	38	7.34	0.50	0.0222	0.012	1.852		
			0.18	279	38	7.34	1.90	0.0844	0.020	4.222		
D2			0.18	279	38	7.34	0.41	0.0182	0.013	1.402		
			0.18	279	38	7.34	0.94	0.0418	0.024	1.741		
			0.18	279	38	7.34	1.70	0.0756	0.033	2.290		
D3			0.18	279	38	7.34	0.46	0.0204	0.011	1.859		
			0.18	279	38	7.34	1.10	0.0489	0.016	3.056		
M1			0.18	279	38	7.34	0.91	0.0404	0.023	1.758	2.27	0.931
Wood 1986	10	2.5	0.22	610	57	10.7	0.39	0.0142	0.010	1.418		
Tower			0.22	610	57	10.7	0.61	0.0222	0.017	1.305		
			0.22	610	57	10.7	0.81	0.0295	0.020	1.473		
			0.22	610	57	10.7	1.10	0.0400	0.026	1.538	1.43	0.099
Otani 1972												
D1	3	1	0.22	914	76	12	0.24	0.0087	0.006	1.385		
			0.22	914	76	12	0.40	0.0145	0.011	1.322		
			0.22	914	76	12	0.53	0.0193	0.015	1.285		
			0.22	914	76	12	0.84	0.0305	0.024	1.273		
			0.22	914	76	12	1.40	0.0509	0.030	1.697		
			0.22	914	76	12	3.20	0.1164	0.038	3.062		
D2			0.22	914	76	12	0.86	0.0313	0.024	1.303		
			0.22	914	76	12	1.10	0.0400	0.032	1.250		
			0.22	914	76	12	1.20	0.0436	0.037	1.179		
			0.22	914	76	12	3.40	0.1236	0.044	2.810		
D3			0.22	914	76	12	0.61	0.0222	0.020	1.109		
			0.22	914	76	12	1.10	0.0400	0.032	1.250		
			0.22	914	76	12	0.93	0.0338	0.037	0.914		
			0.22	914	76	12	2.10	0.0764	0.042	1.818	1.55	0.631
Abrams	10	2.5	0.3	305	38	8.03	0.51	0.0136	0.012	1.133		
1979 FW1			0.3	305	38	8.03	1.70	0.0453	0.017	2.667		
			0.3	305	38	8.03	2.50	0.0667	0.030	2.222		

Table 6 cont.: Structural Index, ratios of calculated to measured mean drift ratio (flat rate method), L/h ratio

Structure	Stories	Correction Factor	Structural Index SI	Girder length, L [mm]	Girder height, h [mm]	L / h	Peak acceleration [g] (1st acc.)	MDR (Mean drift ratio) calculated	MDR (Mean drift ratio) measured	MDR calc / MDR measured	Average MDRc/MDRm ratio per test structure	Standard deviation
Abrams 1979												
FW2			0.3	305	38	8.03	0.48	0.0128	0.012	1.067		
			0.3	305	38	8.03	0.92	0.0245	0.019	1.291		
			0.3	305	38	8.03	1.10	0.0293	0.024	1.222		
FW3			0.3	305	38	8.03	0.42	0.0112	0.007	1.514		
			0.3	305	38	8.03	0.96	0.0256	0.021	1.219		
			0.3	305	38	8.03	1.10	0.0293	0.025	1.173		
FW4			0.3	305	38	8.03	0.47	0.0125	0.008	1.586		
			0.3	305	38	8.03	0.94	0.0251	0.020	1.253		
			0.3	305	38	8.03	1.30	0.0347	0.028	1.238	1.47	0.489
Moehle 1980												
FSW	9	2.3	0.38	305	38	8.03	0.34	0.0072	0.010	0.746		
			0.38	305	38	8.03	0.59	0.0124	0.017	0.731		
			0.38	305	38	8.03	1.10	0.0232	0.032	0.724		
FHW			0.38	305	38	8.03	0.41	0.0086	0.010	0.863		
			0.38	305	38	8.03	0.48	0.0101	0.017	0.594	*TPU	
			0.38	305	38	8.03	0.73	0.0154	0.029	0.530		
FFW			0.38	305	38	8.03	0.32	0.0067	0.011	0.612		
			0.38	305	38	8.03	0.55	0.0116	0.019	0.609		
			0.38	305	38	8.03	0.80	0.0168	0.031	0.543	0.66	0.110
Lybas 1977												
D1	6	1.5	0.7	279	57	4.89	0.12	0.0014	0.001	1.247		
			0.7	279	57	4.89	0.22	0.0025	0.003	0.898		
			0.7	279	57	4.89	0.50	0.0057	0.005	1.099		
			0.7	279	57	4.89	1.10	0.0126	0.009	1.352		
			0.7	279	57	4.89	2.20	0.0251	0.020	1.257		
D2			0.7	279	38	7.34	1.30	0.0149	0.008	1.790		
			0.7	279	38	7.34	3.60	0.0411	0.025	1.646		
D3			0.7	279	38	7.34	1.10	0.0126	0.009	1.479		
			0.7	279	38	7.34	2.10	0.0240	0.019	1.263		
D4			0.7	279	38	7.34	1.10	0.0126	0.009	1.352		
			0.7	279	38	7.34	2.40	0.0274	0.021	1.306		
D5			0.7	279	38	7.34	1.10	0.0126	0.009	1.413		
			0.7	279	38	7.34	2.10	0.0240	0.022	1.091	1.32	0.234
Wolfgram												
1984 NS1	7	1.8	13.93	600	50	12.0	0.59	0.0003	0.008	0.040		
			13.93	600	50	12.0	1.80	0.0010	0.022	0.047		
NS2			13.93	600	50	12.0	0.59	0.0003	0.007	0.048		
			13.93	600	50	12.0	1.00	0.0006	0.013	0.043		
			13.93	600	50	12.0	1.60	0.0009	0.013	0.069		
			13.93	600	50	12.0	1.50	0.0009	0.026	0.033	*TPU	
NS3			13.93	600	50	12.0	0.49	0.0003	0.007	0.043		
			13.93	600	50	12.0	0.82	0.0005	0.011	0.044		
			13.93	600	50	12.0	1.50	0.0009	0.011	0.077		
			13.93	600	50	12.0	1.50	0.0009	0.023	0.037	0.05	0.014

*TPU : For these tests, the target period
method underestimated the drift.

Table 6 cont.: Structural Index, ratios of calculated to measured mean drift ratio (flat rate method), L/h ratio

Target Period Method

Structure / Report	Frame	Number of Stories	ScaleFactor for Length, SL	ScaleFactor for Time, ST	Initial period, Ti [sec.]	Modal participation factor, MPF	Amplification factor, Aa	Characteristic period of design ground motion, Tg	Design peak ground acceleration, a	k factor	Max. measured roof drift, [mm]	Expected max. drift, D [mm]	Drift calc / drift measured	Average of Dc/Dm ratio per test structure	Standard deviation
1) Oliva, Clough		2	7/10	1	0.28	1.3	3.75	0.72	0.306	205.30	39.12	105.68	2.70		
1987				1	0.28	1.3	3.75	0.72	0.684	458.91	53.85	236.24	4.39		
				1	0.28	1.3	3.75	0.72	0.798	535.40	51.82	275.61	5.32	4.14	1.326
2) Oliva		2	7/10	1	0.23	1.3	3.75	0.72	0.062	41.60	12.78	17.59	1.38		
1980				1	0.23	1.3	3.75	0.72	0.061	40.93	12.65	17.31	1.37		
				1	0.23	1.3	3.75	0.72	0.685	459.58	128.78	194.33	1.51		
				1	0.23	1.3	3.75		1.49	0.00	132.84	-	-		
				1	0.23	1.3	3.75		1.37	0.00	133.35	-	-		
				1	0.23	1.3	3.75	0.72	0.711	477.03	128.52	201.71	1.57	1.46	0.10
3) Hidalgo	N1*	2	7/10	1	0.23	1.3	3.75	0.72	0.07	46.96	2.87	19.86	6.92		
1975	N2			1	0.23	1.3	3.75	0.72	0.11	73.80	5.44	31.21	5.74		
	N3			1	0.23	1.3	3.75	0.72	0.22	147.60	14.48	62.41	4.31		
	N4			1	0.23	1.3	3.75	0.72	0.24	161.02	18.57	68.09	3.67		
	W1			1	0.23	1.3	3.75	0.72	0.07	46.96	8.71	19.86	2.28		
	W2			1	0.23	1.3	3.75	0.72	0.10	67.09	21.77	28.37	1.30		
	W3			1	0.23	1.3	3.75	0.72	0.22	147.60	38.74	62.41	1.61		
	W4			1		1.3			--						
	W5			1	0.23	1.3	3.75	0.72	0.30	201.28	38.74	85.11	2.20		
	W6			1	0.23	1.3	3.75	0.72	0.44	295.21	73.71	124.83	1.69		
	W7			1	0.23	1.3	3.75	0.72	0.30	201.28	86.33	85.11	0.99		
	W8			1	0.23	1.3	3.75	0.72	0.22	147.60	55.85	62.41	1.12		
	W9			1	0.23	1.3	3.75	0.55	0.15	76.88	40.69	32.51	0.80		
	W10			1	0.23	1.3	3.75	0.55	0.46	235.76	92.51	99.69	1.08		
	R1			1	0.23	1.3	3.75	0.72	0.08	53.67	8.18	22.70	2.77		
	R2			1	0.23	1.3	3.75	0.72	0.41	275.08	75.06	116.32	1.55		
	R3			1	0.23	1.3	3.75	0.72	0.64	429.39	113.84	181.57	1.59	2.48	1.80
5) Kitagawa	DR 10	2	1/2	0.5	0.33	1.3	3.75	0.475	0.216	95.68	12.3	58.05	4.72		
1984	DR 20	-		0.5	0.46	1.3	3.75	0.475	0.566	250.50	50.1	211.84	4.23	4.47	0.347

Table 7: Calculation of expected drift using the target period method

Structure / Report	Frame	Number of Stories	ScaleFactor for Length, SL	ScaleFactor for Time, ST	Initial period, Ti [sec.]	Modal participation factor, MPF	Amplification factor, Aa	Characteristic period of design ground motion, Tg	Design peak ground acceleration, a	k factor	Max. measured roof drift, [mm]	Expected max. drift, D [mm]	Drift calc / drift measured	Average of Dc/Dm ratio	Standard deviation
6) Eto, Hiroaki, Takeda		3	1/6	1		1.3									
YD1	YD1	-		1	0.29	1.3	3.75	1.14	0.271	288.14	95.9	153.62	1.60		
YD2	YD2	-		1	0.24	1.3	3.75	0.72	0.197	132.04	32.0	58.26	1.82	1.71	0.155
YD2	YD2			1		1.3					scale out				
7) Minowams, Ohtani, Ogawa: Type A	Type A	1	1/1	1	0.24	1.3	3.75	1.14	0.816	866.58	144	379.33	2.63		
1994	Type A			1	0.24	1.3	3.75	1.14	0.744	790.75	165	346.14	2.10		
Type B	Type B	1		1	0.23	1.3	3.75	1.14	0.642	682.43	109	285.14	2.62	2.45	0.305
9) Vintzileou, Yong Lu, Zhang															
BF1	BF1	6	1/5.5	0.43	0.41	1.3	3.75	0.23	0.1	21.85	7.5	16.47	2.20		
1998	BF1			0.43	0.41	1.3	3.75	0.23	0.3	65.56	27.0	49.42	1.83		
	BF1			0.43	0.41	1.3	3.75	0.23	0.6	131.12	44.3	98.84	2.23		
	BF1			0.43	0.41	1.3	3.75	0.23	0.9	196.68	60.7	148.25	2.44		
	BF1			0.43	0.41	1.3	3.75	0.23	1.2	262.24	88.6	197.67	2.23		
SWF	SWF	6	1/5.5	0.43	0.42	1.3	3.75	0.23	0.1	21.83	2.27	16.86	7.43		
	SWF			0.43	0.42	1.3	3.75	0.23	0.3	65.56	10.23	50.62	4.95		
	SWF			0.43	0.42	1.3	3.75	0.23	0.6	131.12	22.73	101.25	4.45		
	SWF			0.43	0.42	1.3	3.75	0.23	0.9	196.68	46.59	151.87	3.26		
	SWF			0.43	0.42	1.3	3.75	0.23	1.2	262.24	69.32	202.49	2.92	3.39	1.749
10) Moehle, Sozen 1980	FNW	9	0.08	0.40	0.19	1.3	3.75	0.22	0.39	79.95	19.50	27.93	1.43		
				0.40	0.19	1.3	3.75	0.22	0.78	159.90	44.00	55.86	1.27		
	FSW			0.40	0.19	1.3	3.75	0.22	0.34	69.70	18.20	24.35	1.34		
				0.40	0.19	1.3	3.75	0.22	0.59	120.95	40.00	42.25	1.06		
	FHW			0.40	0.19	1.3	3.75	0.22	0.41	84.05	17.60	29.36	1.67		
				0.40	0.19	1.3	3.75	0.22	0.48	98.40	40.00	34.37	0.86		
	FFW			0.40	0.19	1.3	3.75	0.22	0.32	65.60	17.30	22.92	1.32		
				0.40	0.19	1.3	3.75	0.22	0.55	112.75	44.00	39.39	0.90	1.23	0.276

Table 7 cont.: Calculation of expected drift using the target period method

Structure / Report	Number of Stories	Initial period, Ti [sec.]	Modal participation factor, MPF	Amplification factor, Aa	Characteristic period of design ground motion, Tg	Design peak ground acceleration, a	k factor	Max. measured roof drift, [mm]	Expected max. drift, D [mm]	Drift calc / drift measured	Average of Dc/Dm ratio per test structure	Standard deviation
Bracci et al. B1	3	0.4	1.3	3.75	0.35	0.2	65.229	33.5	47.969	1.432		
		0.4	1.3	3.75	0.35	0.3	97.843	59.7	71.953	1.205	1.319	0.160
Schulz 1985 SS1	10	0.21	1.3	3.75	0.22	0.35	71.752	25	27.702	1.108		
		0.21	1.3	3.75	0.22	0.34	69.702	22	26.910	1.223		
		0.21	1.3	3.75	0.22	0.53	108.652	32	41.949	1.311		
SS2		0.21	1.3	3.75	0.22	0.35	71.752	23	27.702	1.204		
		0.21	1.3	3.75	0.22	0.4	82.002	28	31.659	1.131		
		0.21	1.3	3.75	0.22	0.36	73.802	22	28.493	1.295		
		0.21	1.3	3.75	0.22	0.66	135.303	38	52.238	1.375		
		0.21	1.3	3.75	0.22	0.99	202.954	51	78.357	1.536		
		0.21	1.3	3.75	0.22	1.3	266.506	85.84	102.893	1.199	1.265	0.133
Filiatraut '98 R2	2	0.36	1.3	3.75	0.5	0.21	97.843	48.0	64.758	1.349		
		0.36	1.3	3.75	0.5	0.42	195.686	95.0	129.515	1.363		
R4		0.28	1.3	3.75	0.5	0.21	97.843	45.0	50.367	1.119		
		0.28	1.3	3.75	0.5	0.42	195.686	72.0	100.734	1.399	1.308	0.127
Holiday Inn Van Nuys	7	0.9	1.3	3.75	0.4	0.47	175.186	240.3	289.867	1.206	1.206	-
Eberhard 1989 ES1	9	0.17	1.3	3.75	0.22	0.36	73.802	18	23.066	1.281		
		0.17	1.3	3.75	0.22	0.52	106.602	31	33.318	1.075		
		0.17	1.3	3.75	0.22	0.62	127.103	43	39.725	0.924		
ES2		0.17	1.3	3.75	0.22	0.35	71.752	17	22.425	1.319		
		0.17	1.3	3.75	0.22	0.52	106.602	32	33.318	1.041		
		0.17	1.3	3.75	0.22	0.61	125.053	41	39.084	0.953	1.099	0.166
Healy 1978 MF1	10	0.2	1.3	3.75	0.22	0.4	82.002	24	30.152	1.256		
		0.2	1.3	3.75	0.22	0.93	190.654	51	70.103	1.375		
		0.2	1.3	3.75	0.22	1.4	287.006	68	105.531	1.552	1.394	0.149
Moehle 1978 MF2	10	0.21	1.3	3.75	0.22	0.38	77.902	24	30.076	1.253		
		0.21	1.3	3.75	0.22	0.83	170.154	44	65.693	1.493		
		0.21	1.3	3.75	0.22	1.3	266.506	58	102.893	1.774	1.507	0.261

Table 8: Calculation of expected drift using the target period method

Structure / Report	Number of Stories	Initial period, Ti [sec.]	Modal participation factor, MPF	Amplification factor, Aa	Characteristic period of design ground motion, Tg	Design peak ground acceleration, a	k factor	Max. measured roof drift, [mm]	Expected max. drift, D [mm]	Drift calc / drift measured	Average of Dc/Dm ratio per test structure	Standard deviation
Cecen 1979 H1	10	0.20	1.3	3.75	0.22	0.36	73.802	29	27.137	0.936		
		0.20	1.3	3.75	0.22	0.84	172.204	52	63.319	1.218		
		0.20	1.3	3.75	0.22	1.6	328.007	100.8	120.607	1.197		
H2		0.21	1.3	3.75	0.22	0.17	34.851	8.9	13.455	1.507		
		0.21	1.3	3.75	0.22	0.33	67.651	18	26.119	1.451		
		0.21	1.3	3.75	0.22	0.49	100.452	24	38.783	1.616		
		0.21	1.3	3.75	0.22	0.47	96.352	26	37.200	1.431		
		0.21	1.3	3.75	0.22	0.72	147.603	39	56.987	1.461		
		0.21	1.3	3.75	0.22	1	205.004	58	79.148	1.365		
		0.21	1.3	3.75	0.22	2.6	533.011	103.1	205.785	1.997	1.418	0.281
Wood 1986	10	0.16	1.3	3.75	0.22	0.44	90.202	23	26.533	1.154		
Stepped		0.16	1.3	3.75	0.22	0.61	125.053	31	36.785	1.187		
		0.16	1.3	3.75	0.22	0.9	184.504	37	54.273	1.467	1.269	0.172
Aristizabal 1979												
D1	10	0.19	1.3	3.75	0.22	0.5	102.502	28	35.805	1.279		
		0.19	1.3	3.75	0.22	1.9	389.508	46	136.059	2.958		
D2		0.18	1.3	3.75	0.29	0.41	110.796	29	36.665	1.264		
		0.18	1.3	3.75	0.22	0.94	192.704	54	63.771	1.181		
		0.18	1.3	3.75	0.22	1.7	348.508	75.57	115.330	1.526		
D3		0.19	1.3	3.75	0.29	0.46	124.307	24	43.422	1.809		
		0.19	1.3	3.75	0.29	1.1	297.256	38	103.835	2.732		
M1		0.19	1.3	3.75	0.22	0.91	186.554	52	65.165	1.253	1.750	0.708
Wood 1986	10	0.16	1.3	3.75	0.22	0.39	79.952	22	23.518	1.069		
Tower		0.16	1.3	3.75	0.22	0.61	125.053	36	36.785	1.022		
		0.16	1.3	3.75	0.22	0.81	166.054	43	48.846	1.136		
		0.16	1.3	3.75	0.22	1.1	225.505	56	66.334	1.185	1.103	0.072
Otani 1972 D1	3	0.15	1.3	3.75	0.22	0.24	49.201	8.40	13.568	1.616		
		0.15	1.3	3.75	0.22	0.4	82.002	14	22.614	1.615		
		0.15	1.3	3.75	0.22	0.53	108.652	20	29.963	1.498		
		0.15	1.3	3.75	0.22	0.84	172.204	31	47.489	1.532		
		0.15	1.3	3.75	0.22	1.4	287.006	40	79.148	1.979		
		0.15	1.3	3.75	0.29	3.2	864.746	50.65	238.472	4.708		
D2		0.15	1.3	3.75	0.22	0.86	176.304	32	48.620	1.519		
		0.15	1.3	3.75	0.22	1.1	225.505	42.66	62.188	1.458		
		0.15	1.3	3.75	0.29	1.2	324.280	49.32	89.427	1.813		
		0.15	1.3	3.75	0.29	3.4	918.793	58.65	253.377	4.320		
D3		0.15	1.3	3.75	0.22	0.61	125.053	27	34.486	1.277		
		0.15	1.3	3.75	0.22	1.1	225.505	42.66	62.188	1.458		
		0.15	1.3	3.75	0.29	0.93	251.317	49.32	69.306	1.405		
		0.15	1.3	3.75	0.29	2.1	567.490	55.99	156.498	2.795	2.071	1.102

Table 8 cont.: Calculation of expected drift using the target period method

Structure / Report	Number of Stories	Natural period, Ti [sec.]	Modal participation factor, MPF	Amplification factor, Aa	Characteristic period of design ground motion, Tg	Design peak ground acceleration, a	K factor	Max. measured roof drift, [mm]	Expected max. drift, D [mm]	Drift calc / drift measured	Average of Dc/Dm ratio per test structure	Standard deviation
Abrams 1979 FW1	10	0.19	1.3	3.75	0.22	0.51	104.552	28	36.521	1.304		
		0.19	1.3	3.75	0.22	1.7	348.508	38	121.737	3.204		
		0.19	1.3	3.75	0.22	2.5	512.511	68	179.026	2.633		
FW2		0.17	1.3	3.75	0.22	0.48	98.402	28	30.755	1.098		
		0.17	1.3	3.75	0.22	0.92	188.604	43	58.947	1.371		
		0.17	1.3	3.75	0.22	1.1	225.505	56	70.480	1.259		
FW3		0.19	1.3	3.75	0.29	0.42	113.498	17	39.646	2.332		
		0.19	1.3	3.75	0.29	0.96	259.424	48	90.620	1.888		
		0.19	1.3	3.75	0.29	1.1	297.256	58	103.835	1.790		
FW4		0.18	1.3	3.75	0.29	0.47	127.010	18	42.031	2.335		
		0.18	1.3	3.75	0.29	0.94	254.019	46	84.062	1.827		
		0.18	1.3	3.75	0.29	1.3	351.303	65	116.255	1.789	1.902	0.627
Moehle 1980 FSW	9	0.18	1.3	3.75	0.22	0.34	69.702	22	23.066	1.048		
		0.18	1.3	3.75	0.22	0.59	120.953	40	40.026	1.001		
		0.18	1.3	3.75	0.22	1.1	225.505	73	74.625	1.019		
FHW		0.18	1.3	3.75	0.22	0.41	84.052	23	27.815	1.209		
		0.18	1.3	3.75	0.22	0.48	98.402	40	32.564	0.814		
		0.18	1.3	3.75	0.22	0.73	149.653	66	49.524	0.750		
FFW		0.18	1.3	3.75	0.22	0.32	65.601	26	21.709	0.835		
		0.18	1.3	3.75	0.22	0.55	112.752	44	37.313	0.848		
		0.18	1.3	3.75	0.22	0.8	164.004	71	54.273	0.765	0.921	0.155
Lybas 1977 D1	6	0.09	1.3	3.75	0.11	0.12	12.300	-	2.035			
		0.09	1.3	3.75	0.11	0.22	22.550	-	3.731			
		0.09	1.3	3.75	0.11	0.5	51.251	7	8.480	1.211		
		0.09	1.3	3.75	0.11	1.1	112.752	13	18.656	1.435		
		0.09	1.3	3.75	0.11	2.2	225.505	27	37.313	1.382		
D2		0.09	1.3	3.75	0.11	1.3	133.253	-	22.048			
		0.09	1.3	3.75	0.11	3.6	369.008	-	61.057			
D3		0.09	1.3	3.75	0.11	1.1	112.752	12	18.656	1.555		
		0.09	1.3	3.75	0.11	2.1	215.255	25	35.617	1.425		
D4		0.09	1.3	3.75	0.11	1.1	112.752	13	18.656	1.435		
		0.09	1.3	3.75	0.11	2.4	246.005	29	40.705	1.404		
D5		0.09	1.3	3.75	0.11	1.1	112.752	12	18.656	1.555		
		0.09	1.3	3.75	0.11	2.1	215.255	30	35.617	1.187	1.399	0.128

Table 8 cont.: Calculation of expected drift using the target period method

Structure / Report	Number of Stories	Natural period, Ti [sec.]	Modal participation factor, MPF	Amplification factor, Aa	Characteristic period of design ground motion, Tg	Design peak ground acceleration, a	k factor	Max. measured roof drift, [mm]	Expected max. drift, D [mm]	Drift calc / drift measured	Average of Dc/Dm ratio per test structure	Standard deviation
Wolfgram 1984 NS1	7	0.09	1.3	3.75	0.19	0.59	104.459	18	17.284	0.960		
		0.09	1.3	3.75	0.19	1.8	318.689	47	52.731	1.122		
NS2		0.09	1.3	3.75	0.19	0.59	104.459	15	17.284	1.152		
		0.09	1.3	3.75	0.19	1	177.049	29	29.295	1.010		
		0.09	1.3	3.75	0.19	1.6	283.279	29	46.872	1.616		
		0.09	1.3	3.75	0.19	1.5	265.574	56	43.943	0.785		
NS3		0.09	1.3	3.75	0.19	0.49	86.754	14	14.355	1.025		
		0.09	1.3	3.75	0.19	0.82	145.180	23	24.022	1.044		
		0.09	1.3	3.75	0.19	1.5	265.574	24	43.943	1.831		
		0.09	1.3	3.75	0.19	1.5	265.574	50	43.943	0.879	1.143	0.328

Table 8 cont.: Calculation of expected drift using the target period method

		Flat-rate method:		Target period method:			
		MDR: ratio of measured calculated to MDR		Drift: ratio of calculated to measured drift		Peak ground acceleration	
Structure	No. of stories	Average per test structure	Standard deviation per test structure	Average per test structure	Standard deviation per test structure	Maximum per test structure [g]	Minimum per test structure [g]
Minowams et al. 1994	1	0.817	0.113	2.449	0.305	0.816	0.642
Oliva, Clough 1987	2	2.434	0.781	4.136	1.326	0.798	0.306
Oliva 1980	2	1.360	0.568	1.456	0.100	1.490	0.061
Hidalgo 1975	2	1.720	1.211	2.476	1.800	0.64	0.07
Kitagawa 1984	2	0.961	0.296	4.474	0.347	0.566	0.216
Filiatraut 1998	2	1.052	0.149	1.308	0.127	0.42	0.21
Eto et al. 1980	3	4.959	2.595	1.711	0.155	0.308	0.197
Bracci et al.	3	2.011	0.244	1.319	0.160	0.30	0.20
Otani 1972	3	1.547	0.631	2.071	1.102	3.40	0.24
Vintzileou et al. 1998	6	0.934	0.302	3.394	1.749	1.20	0.10
Lybas 1977	6	1.322	0.234	1.399	0.128	3.60	0.12
Wolfgram 1984	7	0.048	0.014	1.143	0.328	1.50	0.49
Holiday Inn Van Nuys	7	2.840	-	1.206	-	0.47	-
Moehle, Sozen 1980	9	0.771	0.173	1.230	0.276	0.78	0.32
Eberhard 1989	9	1.975	0.302	1.099	0.166	0.62	0.35
Healy 1978	10	2.387	0.268	1.394	0.149	1.40	0.40
Moehle 1978	10	2.458	0.431	1.507	0.261	1.30	0.38
Cecen 1979	10	2.228	0.425	1.418	0.281	2.60	0.17
Wood 1986 Stepped	10	2.141	0.315	1.269	0.172	0.90	0.44
Wood 1986 Tower	10	1.434	0.099	1.103	0.072	1.10	0.39
Aristizabal 1979	10	2.272	0.931	1.750	0.708	1.70	0.41
Abrams 1979	10	1.466	0.489	1.902	0.627	2.50	0.42
Schulz 1985	10	2.733	0.291	1.265	0.133	1.30	0.34
1 to 3 stories		1.676	1.139	2.417	1.503		
6 to 10 stories		1.497	0.875	1.575	0.926		
Underestimation		0.623	0.320	0.866	0.075		
Overestimation		2.093	0.907	1.888	1.162		
All data		1.555	0.968	1.803	1.168		

Table 9: Average and standard deviation for MDR ratios, peak ground acceleration and expected to measured drift

	Average value for all data:	Standard deviation for all data:	Maximum	Minimum
Structural Index, SI	1.12	3.291	13.93	0.01
L/h ratio	11.58	6.204	32.59	4.89

Table 10: Average and standard deviation for structural index and L/h ratio

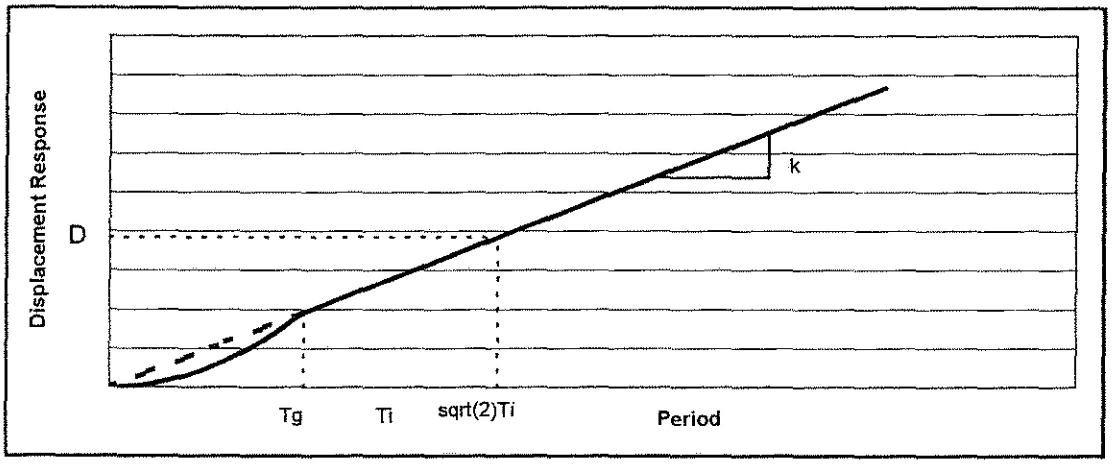


Figure 1: Simplified displacement demand

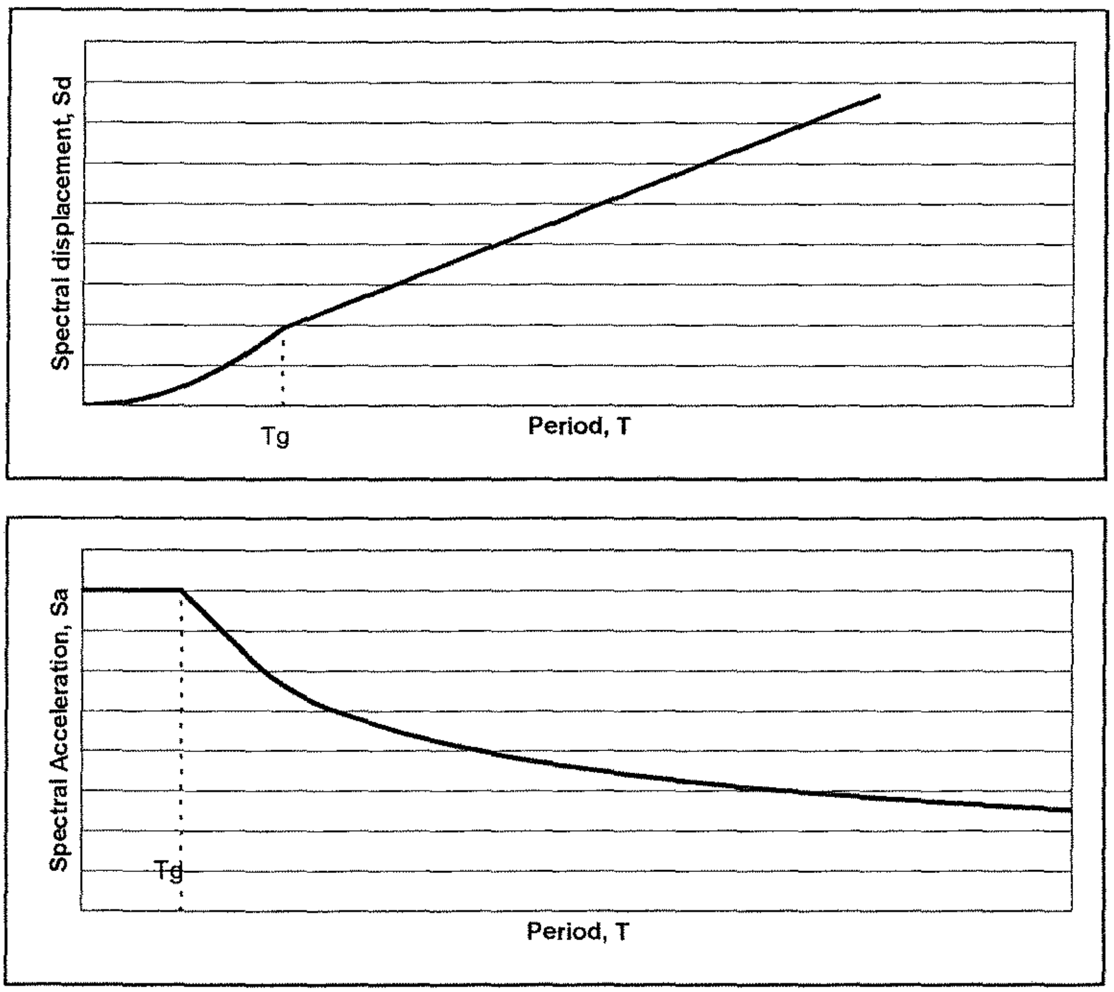


Figure 2: Typical linear response spectrum

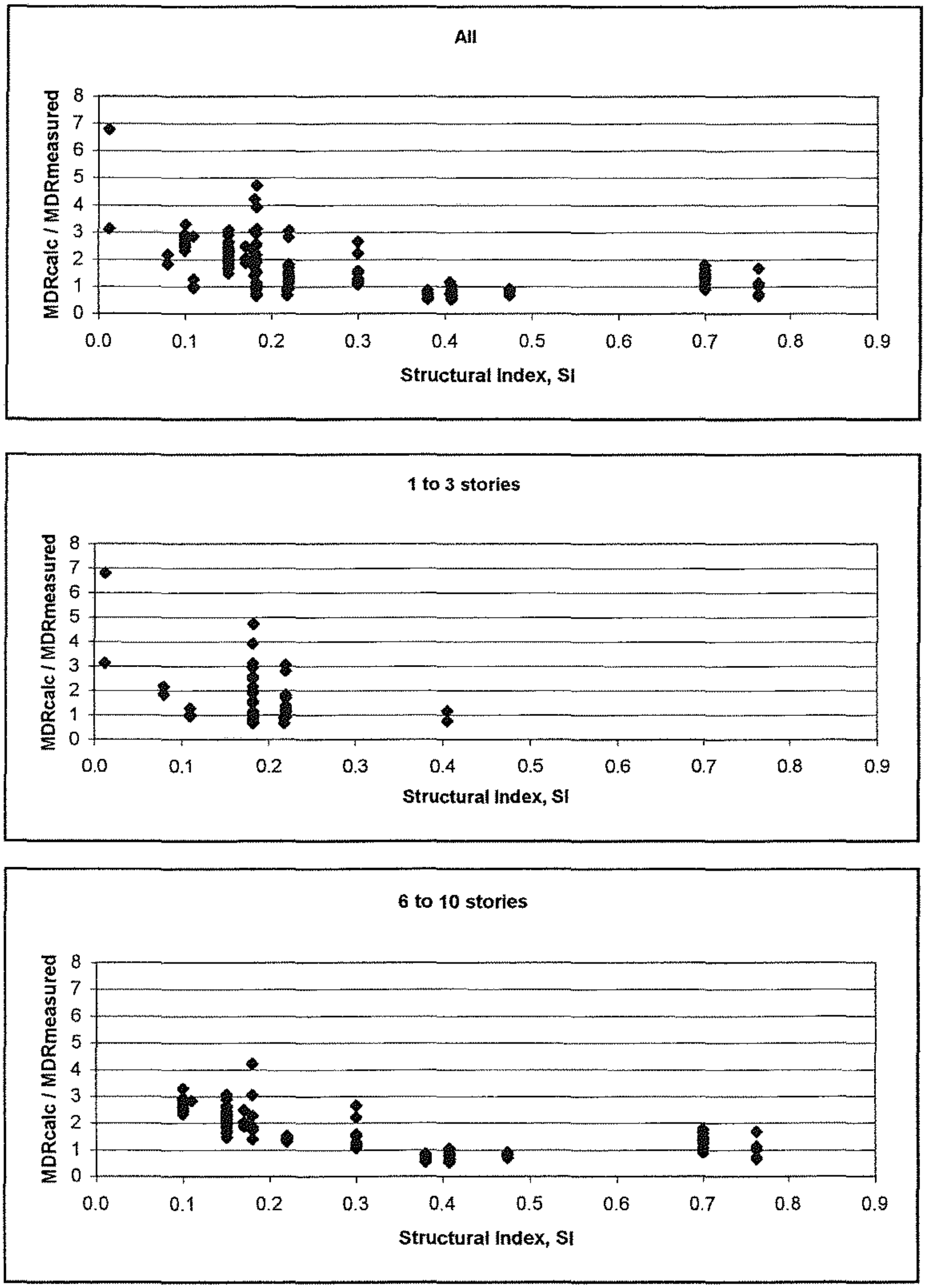


Fig. 3: Structural index vs MDR ratios (calculations according to Hassan and Sozen)

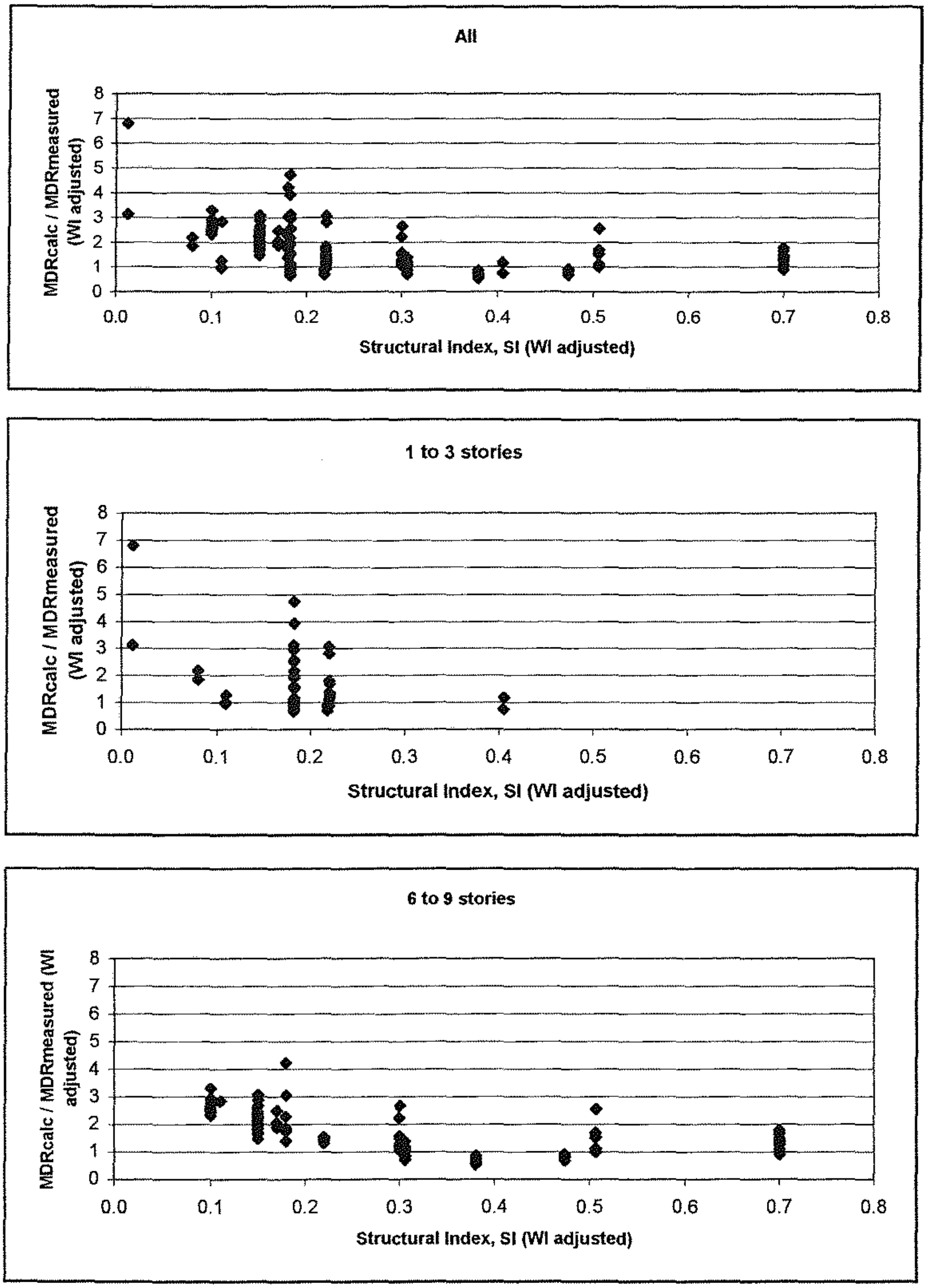


Fig. 4: Structural index vs MDR ratios (calculations adjusted using WI * 0.5)

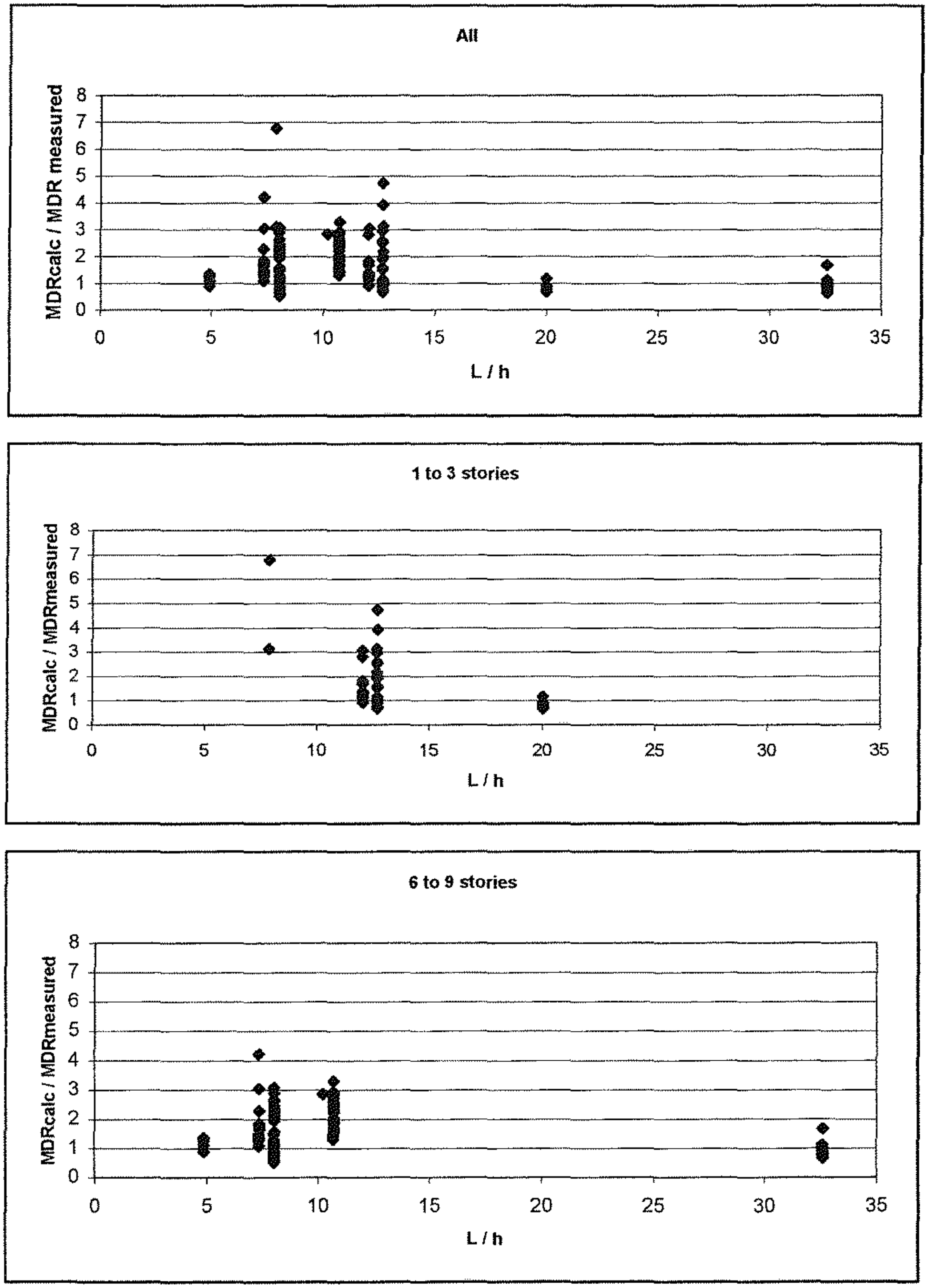


Fig. 5: L to h ratio vs MDR ratios (calculations according to Hassan and Sozen)

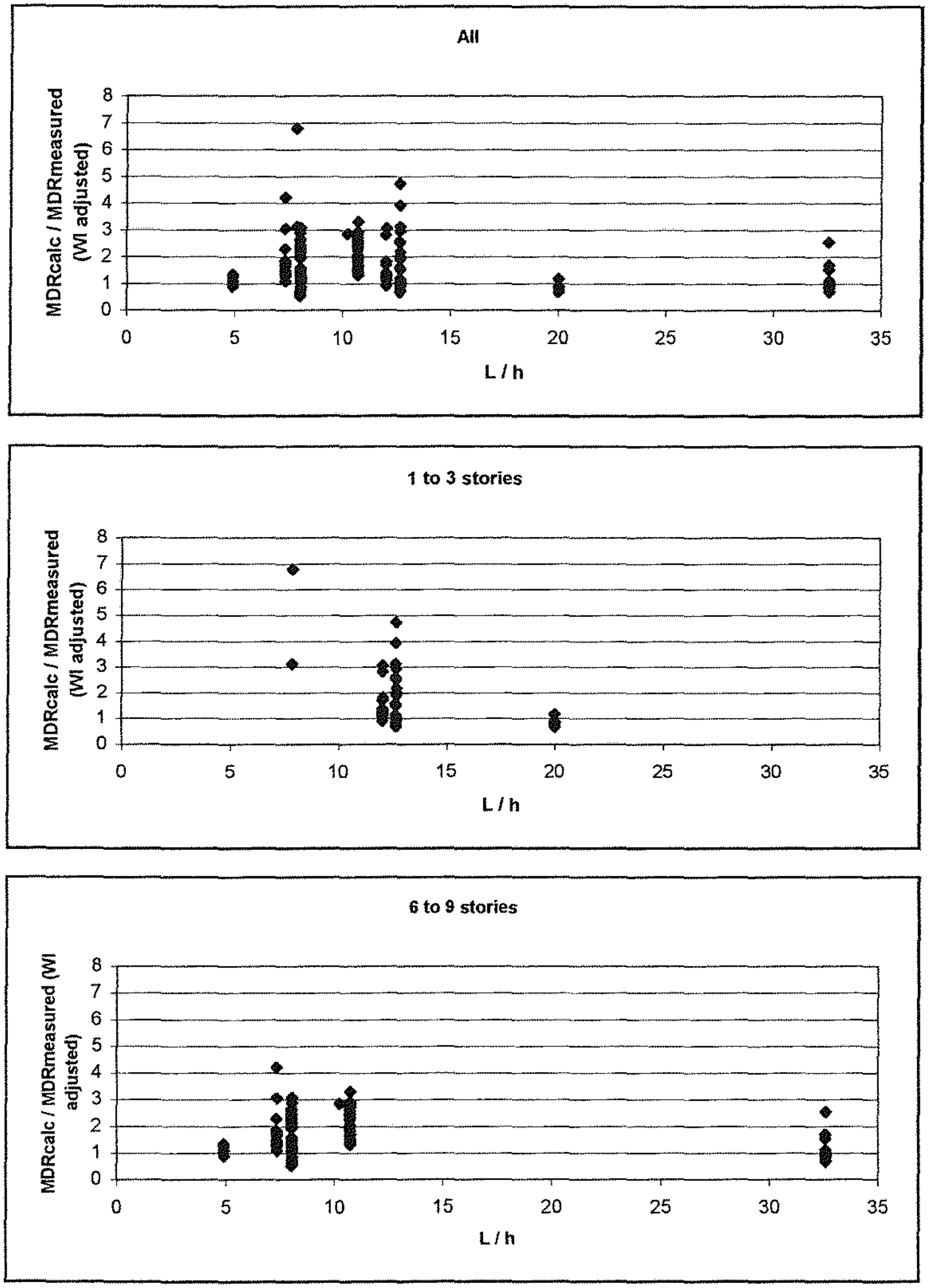


Fig. 6: L to h ratio vs MDR ratios (calculations adjusted using WI * 0.5)

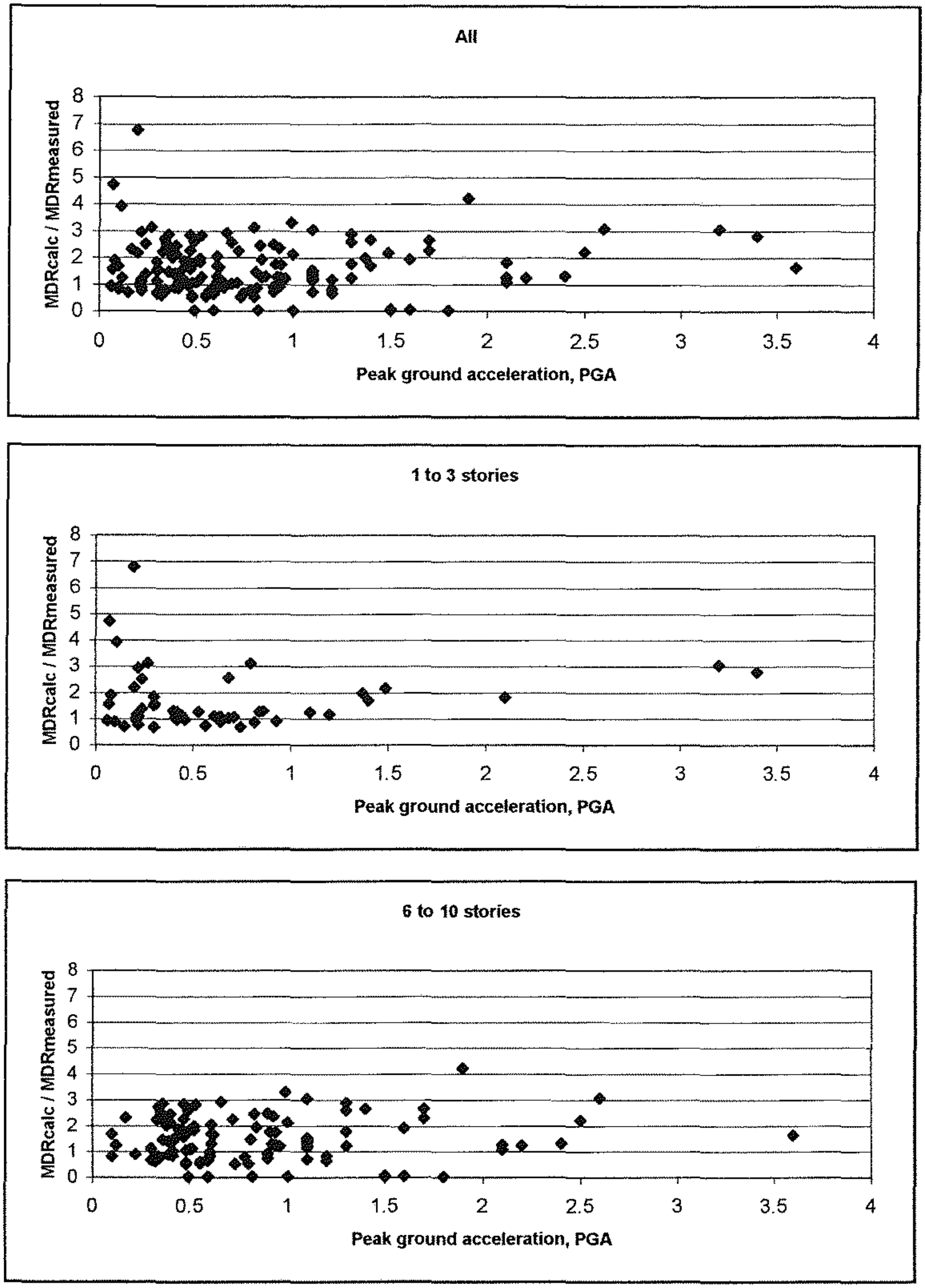
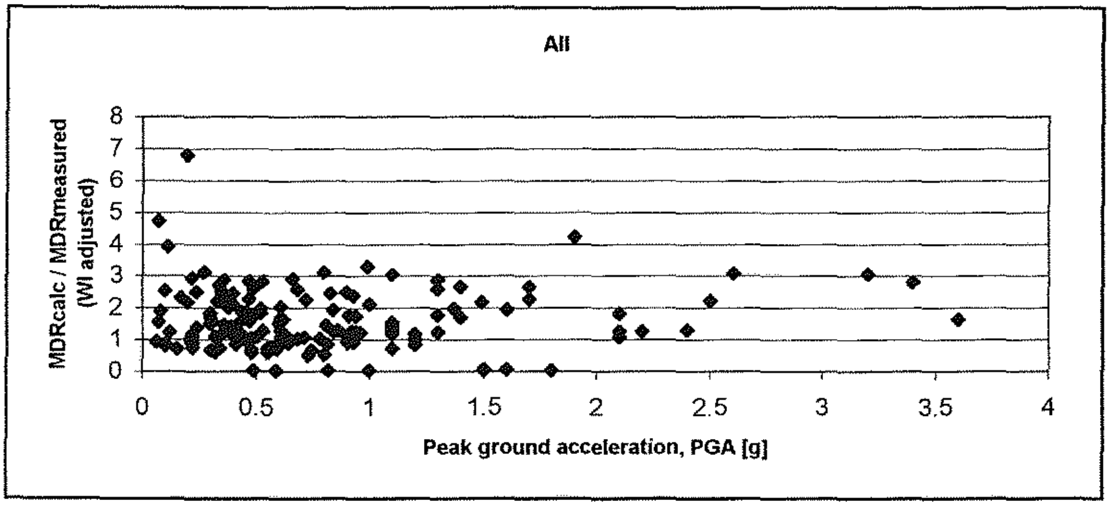
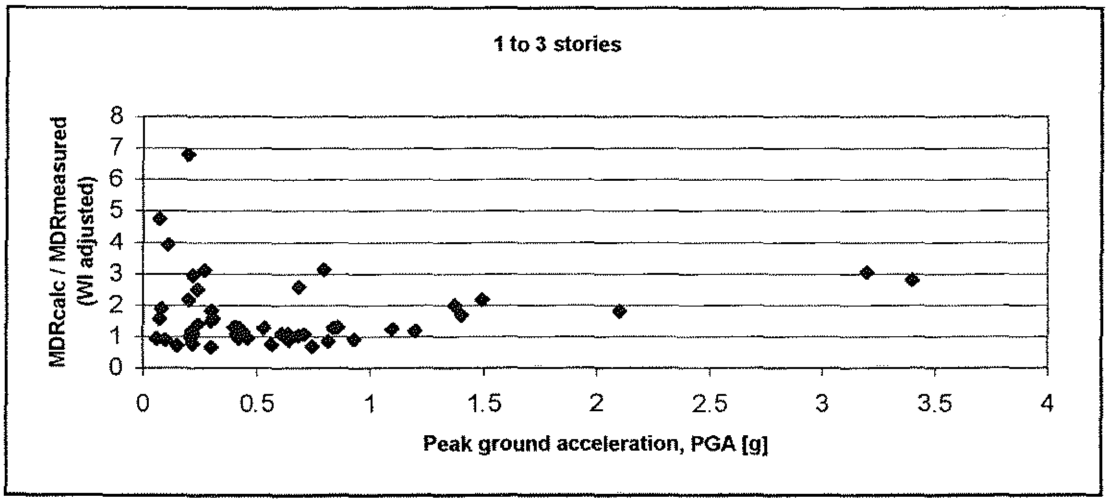


Figure 7: Peak ground acceleration vs MDR ratios (calc. according to Hassan and Sozen)





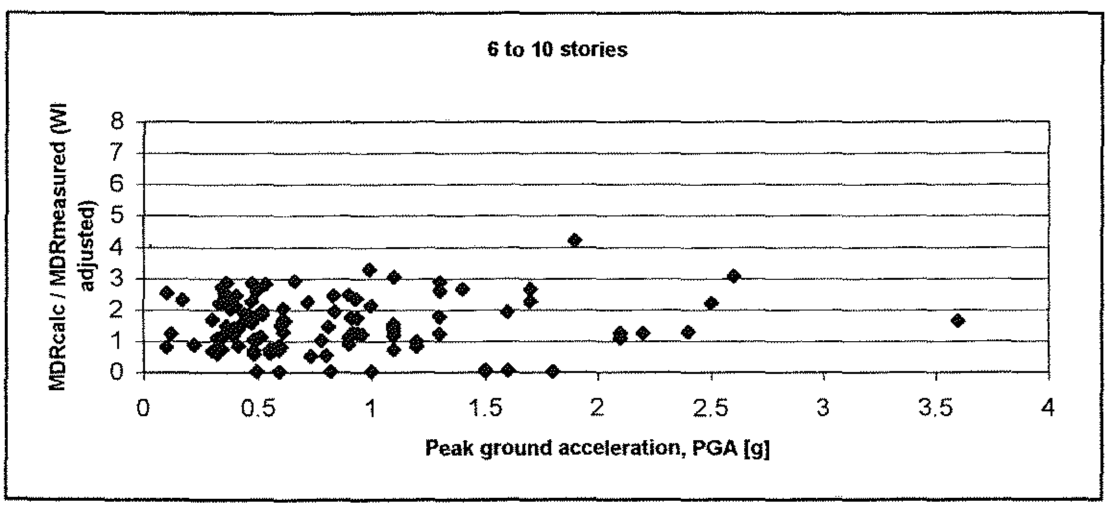


Figure 8: Peak ground acceleration vs MDR ratios (calculations with adjusted equation for WI)

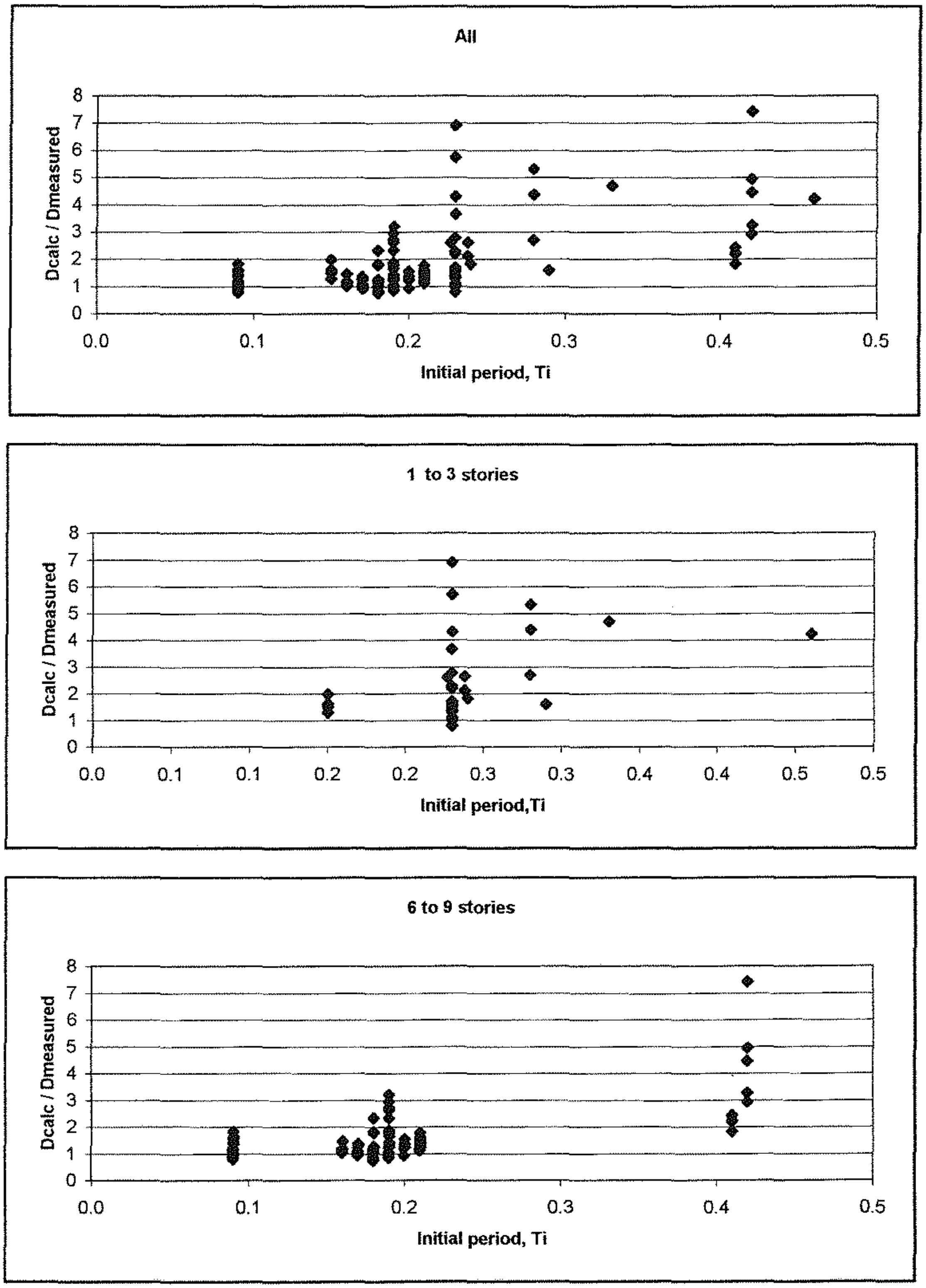


Fig. 9: Initial period vs drift ratios (Target Period Method)

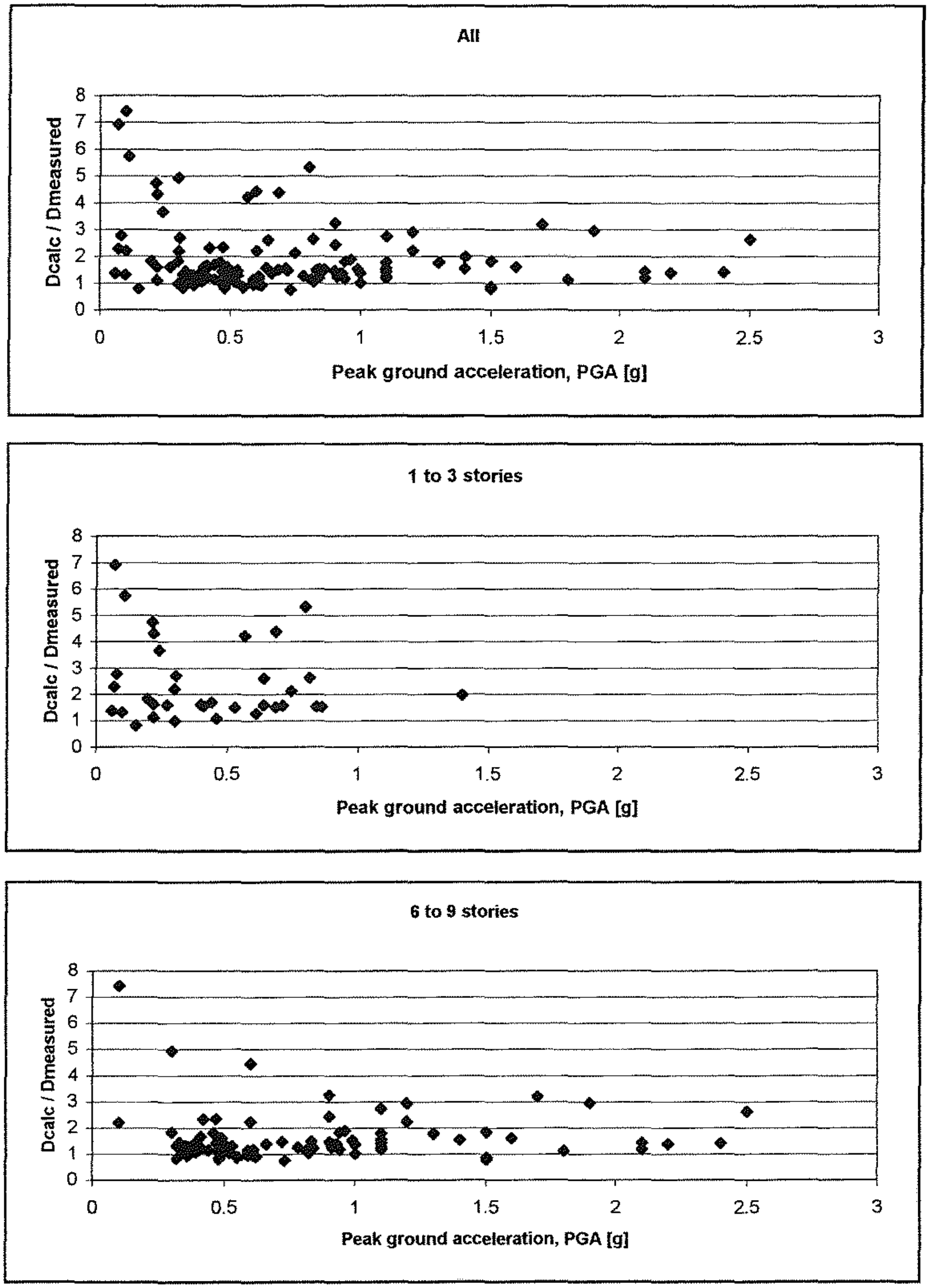


Fig. 10: Peak ground acceleration vs drift ratio

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